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Topics covered include descriptions of applications of the SWMM and of STORM (Storage, Treatment Overflow Runoff Model) in planning, design and infiltration/inflow analysis, a verification study of the Illinois Urban Drainage Area Simulator (ILLUDAS), presentation of a new continuous receiving water model which can be linked to SWMM and STORM and a paper describing combined sewer overflow facilities planning using a macroscopic model.

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4-5 MAY 1978

Project Officer

Harry C. Torno
Office of Air, Land, and Water Use
Office of Research and Development
U.S. Environmental Protection Agency
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OFFICE OF AIR, LAND, AND WATER USE
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FOREWORD

A major function of the Research and Development programs of the Environmental Protection Agency is to effectively and expeditiously transfer, to the user community, technology developed by those programs. A corollary function is to provide for the continuing exchange of information and ideas between EPA and users, and between the users themselves. The Stormwater Management Model (SWMM) users group, sponsored jointly with Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum.

This report, a compendium of papers presented at the last Users Group meeting, is published in the interest of disseminating to a wide audience the work of Group members.

T. A. Murphy
Deputy Assistant Administrator
Office of Air, Land and Water Use

ABSTRACT

This report includes nine papers, on various model-related topics, delivered at the semi-annual Joint U.S.-Canadian Stormwater Management Model (SWMM) Users Group Meeting, held 4-5 May 1978 in Ottawa, Ontario, Canada.

Topics covered include descriptions of applications of the SWMM and of STORM (Storage, Treatment Overflow Runoff Model) in planning, design and infiltration/inflow analysis, a verification study of the Illinois Urban Drainage Area Simulator (ILLUDAS), presentation of a new continuous receiving water model which can be linked to SWMM and STORM and a paper describing combined sewer overflow facilities planning using a macroscopic model.

DISCLAIMER

This report has been reviewed by the Office of Air, Land and Water Use, Office of Research and Development, U.S. Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

STORMWATER MANAGEMENT DEMONSTRATION
STUDY IN ST. THOMAS, ONTARIO

by W. G. Clarke[1], W. L. Knowles[2], R. W. Kuzyk[3]

INTRODUCTION

The various research programs in stormwater management technology conducted by the Urban Drainage Subcommittee of the Canada/Ontario Agreement on Great Lakes Water Quality have been on-going for a number of years. As an outgrowth of the Subcommittee's work, this study was sponsored by C.M.H.C. to demonstrate the practical application of stormwater management and computer modelling to a representative drainage problem. As a demonstration study, it was very comprehensive, including the application of modelling techniques to flood relief, pollution abatement and receiving water response. The principal objectives of the study were:

1) To demonstrate a comprehensive systems approach to stormwater management applications employing modelling techniques

2) To evaluate alternatives and determine the optimal plan for both flood relief and pollution abatement in the study area

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1. W. G. Clarke, B.Sc., P.Eng., Project Engineer, James F. MacLaren Limited

2. W. L. Knowles, M.A.Sc., P. Eng., Project Director, James F. MacLaren Limited

3. R. W. Kuzyk, M.Sc., P. Eng., Project Manager, James F. MacLaren Limited

As a demonstration study, the techniques employed were to be readily applicable to other locations. Accordingly, a considerable portion of the study was devoted to the development and discussion of new analytical techniques and adapting existing methods for this purpose. It is this aspect of the study which will form the basis of this paper.

The Study Area

The City of St. Thomas has a population of 27,000 and is located on Kettle Creek in Elgin County in southwestern Ontario (see Figure 1). The total area of 1841 hectares is 44% residential, 14% industrial, 6.5% commercial and the remaining 35.5% is open space.

The majority of the storm drainage from the City is conveyed to Kettle Creek or its tributaries via a system of combined sewers, separated storm sewers and natural channels. When tributary flows to the treatment plant exceed .37 m^3/sec, overflows directly to the Creek begin. This is equivalent to about twice average dry weather flow. The secondary treatment plant effluent is discharged to Kettle Creek below the City and currently meets M.O.E. standards.

Approximately 25% of the City area is served by combined sewers while the remainder of the existing development and all new development has separated sewers. Much of the combined sewer system was originally designed as a sanitary system without capacity for storm drainage. It is therefore surcharged and frequently results in extensive basement flooding. A 1968 report recommended a relief scheme based on sewer separation and a number of the proposed storm sewers have been installed.

Receiving Water

Kettle Creek has a watershed area of 44 290 hectares and an average gradient of 0.144%. From the confluence with Dodd's Creek where St. Thomas is located, the Creek meanders widely through a deep flat bottomed valley.

Daily flow records available from the Water Survey of Canada indicate that extremely low flows may occur in the months June through October. Figure 2 illustrates the extreme variation in flow which occurs.

Previous studies of Kettle Creek by the M.O.E. indicated that the water quality was seriously impaired by wastewater discharges and a number of large-scale solutions were proposed including low-flow augmentation, tertiary treatment or a pipeline to Lake Erie carrying either raw sewage to a treatment plant or effluent for disposal.

FLOOD RELIEF ANALYSIS

Criteria

The drainage criteria for use in this study can be summarized in three points:

1. Eliminate basement flooding due to inadequate sewer capacity
2. Minimize damages resulting from surface flooding of private property
3. Minimize inconvenience due to street flooding

Stated in this way, basement flooding is not to occur from insufficient sewer capacity for any storm event while the degree and frequency of surface flooding becomes a design variable. The frequency of the design storm, the capacity of the sewer system, the use of runoff controls, and the depth of ponding in the street and the corresponding costs are all evaluated by the analysis of many alternatives based on the actual local conditions. From this information the specific criteria for that location are selected.

Detailed Study Area

One drainage area (D-2) within the City of St. Thomas as defined in a previous (1968) report, was chosen for this study to develop and screen control measures to relieve basement flooding. The boundaries of this area embrace the "Central Business District" and the predominantly residential areas immediately to the north. The area was selected for analysis because of frequent street ponding and extensive basement flooding problems. The results from this area were extrapolated to the entire City.

Analysis of Alternatives

The range of possible solutions to drainage problems using stormwater management is very large and complex. For this reason, a means of screening the many options available was developed.

In this study, several alternatives were considered which included combinations of the following control measures:

1. Roof leader disconnection in residential areas
2. Parking lot and roof detention storage in commercial and industrial areas
3. System surcharge
4. In-system storage
5. Street storage

Coupled with different design levels and in various combinations, a very large number of alternatives were identified as shown in Table 1. Because of the large number of relief alternatives, the screening of alternatives on a total system basis using detailed computer models was not warranted because of cost and time. For the purpose of screening, two small test areas typical of Area D-2 and representative in terms of land use and drainage characteristics were selected to evaluate the cost effectiveness of different relief alternatives. The first test area was a 2.7 ha residential area while the second was about 2.8 ha within the Central Business District.

The method employed to evaluate different relief alternatives can be described in six stages:

1. Simulate the existing system in D-2 using an historical event and compare the results with measurements or flooding records. The return frequency of historical flooding events is also determined

2. Simulate the runoff hydrographs for the two test areas for 2, 5, 10 and 25-year design storms.

3. Determine the control measures required for different relief alternatives and their corresponding costs for the two test areas.

4. Pro-rate the control measure requirements and the costs of relief alternatives to Area D-2.

5. Evaluate the best alternative(s) based on the cost of the relief systems, effectiveness, impact on pollution abatement, and subjective parameters using a matrix scoring technique.

6. Refine the selected relief alternative(s) by a detailed simulation of the runoff hydrographs and peak flows of the relief system for Area D-2

The Stormwater Management Model (SWMM) was applied to the analysis and design of the relief system. The SWM Runoff model was used to simulate runoff hydrographs from different design storms (based on the 'Chicago Method') and the EXTRANS hydraulic model (an optional subroutine of SWMM) was used to route these hydrographs through the sewer system under both free flow and surcharged conditions.

The storage requirements and pipe sizes for different relief alternatives were estimated manually by a graphical method illustrated in Figure 3. Runoff hydrographs were developed using the SWMM Runoff Block for each combination of test area and design storm. Required pipe sizes or storage requirements were then determined assuming constant outflow rates.

Because of the many alternatives investigated, an accurate cost estimate for each alternative for Area D-2 was not warranted. However, the estimated trunk sewer relief system, based on total separation for a 5-year design storm was used as a reference cost for other trunk sewer alternatives. The reference cost was estimated by simulating the runoff hydrographs and peak flows for the relief system for Area D-2 by the SWMM model. The cost for the trunk sewers for other relief alternatives was pro-rated according to the relationship between the inflows from the test areas for each alternative. The costs for the required lateral pipes and control devices was then added in each case according to the lateral system control alternative and the proportion of residential and commercial land use areas in the area. The cost estimates for selected alternatives are shown in Figure 4.

Within each design level (2-year, 5-year, etc.) the various alternatives were equally effective in terms of preventing basement flooding, but each might result in different degrees of surface flooding. The effectiveness of each alternative was then determined by the design level and street flooding aspects. A decision on the best alternative was made on the basis of cost-effectiveness and the desired level of protection.

The many subjective parameters included in the evaluation of alternatives, such as public sensitivity, pollution abatement and cost factors were considered using a matrix scoring technique. A total score was then developed for each alternative for ranking purposes. This was a useful technique to view each alternative in the light of all related aspects.

On the basis of this analysis, a relief scheme based on complete sewer separation and a 5-year design level was selected.

WATER QUALITY ANALYSIS

General Assumptions

Figure 5 shows the main elements under consideration. In this study, it was not possible to consider all aspects of this system to the same degree. The emphasis was placed on the pollutant loadings to Kettle Creek from the St. Thomas pollution control plant and the City's storm drainage system. The effects of these effluents on the dissolved oxygen levels in the Creek were considered, to the extent that the relative significance of different loadings were identified, and the effectiveness of alternative management options were estimated.

Dry weather conditions were examined independently of wet weather conditions. However, the analysis of wet weather conditions could not proceed unless factors relating to dry weather conditions were also considered. For both situations, a number of simplifying assumptions were required to permit use of the limited available data.

Upstream conditions in periods of low flow had been recorded and were sufficiently uniform for the actual sources to be ignored. Therefore, for the dry weather analysis, only the conditions (i.e. flow and water quality) immediately upstream of St. Thomas were considered. Assumptions were made regarding upstream water quality based on measurements conducted during predominantly dry weather periods by the Ministry of the Environment.

For wet weather analysis, it was also assumed that runoff to the Creek from the more impervious urban area of St. Thomas would precede and be independent of the runoff from the rural upstream areas. During the summer period when critical low flows occur in the Creek, actual runoff from the rural areas was very limited for most storms. Similarly, downstream surface runoff flows were not considered significant, and were, therefore, also ignored. Schematics of the wet and dry weather systems considered are shown in Figures 6a and b.

Wet and Dry Weather Loadings

The STORM model was applied to generate pollutant loads from storm and combined sewered areas of St. Thomas during storm events. The City was lumped into two catchments representing the areas served by storm and combined sewers. These two areas overlap due to partial storm sewer separation for relief purposes.

Figure 6a illustrates the wet weather loadings on Kettle Creek that were considered in the study. Average pollutant concentrations based on published data were initially used in the model, but the results of the monitoring program conducted during the study yielded different values that were subsequently used in the final analysis (see Table 2).

Four parameters were selected for simulation as being representative of the various impacts on both the Creek and Lake Erie. Biochemical oxygen demand (BOD) was considered to determine the effects on dissolved oxygen (DO) levels in the Creek. Ammonia nitrogen was converted to equivalent oxygen demand to obtain an estimate of total oxygen demand (TOD) which is the sum of BOD_5 and the ammonia oxygen demand. Total phosphorous was selected as an indicator of nutrient loadings and suspended solids (SS) were included for sediment loadings.

The treatment plant effluent was assumed to be the only source of pollutants during dry weather. In this analysis, "dry weather" days were defined as those days on which the recorded total precipitation was less than 2.54 mm and the maximum hourly rainfall was less than 1.27 mm. A schematic of the dry weather loadings considered in the study is shown in Figure 6b. To isolate the impact of the loading from St. Thomas, ideal upstream conditions were assumed. The treatment plant effluent quality is given in Table 2. Dissolved oxygen levels of the plant effluent were measured at 4.1 mg/l.

Receiving Water Response

Dissolved oxygen (DO) was the parameter selected to indicate the water quality impact in Kettle Creek. This criterion was selected because previous reports on Kettle Creek water quality indicated that there might be a DO problem and because biochemical oxygen demand (BOD) is one of the most commonly used measures of water quality.

Since complete mixing of the treatment plant discharge with the incoming upstream flow and subsequent plug flow were assumed, analysis of the system was carried out using the well-known Streeter-Phelps equation (see Figure 7).

The Streeter-Phelps equation computes the DO deficit as the difference between oxygen demand processes and oxygen renewal processes in a stream. The main processes are shown schematically in Figure 7. Values of the deoxygenation (K_d) and reoxygenation (K_a) rate constants used in the study were taken from a 1976 M.O.E. report.

For the purpose of the computation of dissolved oxygen levels in dry weather, a constant discharge of 0.18 m^3/s at the treatment plant was assumed. The TOD of this effluent was taken as 28 mg/l (BOD_5 of 17.4 mg/l and ammonia oxygen demand of 10.6 mg/l). In order to isolate the DO deficit related to the St. Thomas treatment plant discharge from that attributable to upstream pollutant sources, it was assumed that upstream of the treatment plant DO was at saturation levels (C_s).

The analysis of stream quality under stormflow conditions was more complicated than under dry weather conditions due to the additional unsteady inputs of flow and pollutants to Kettle Creek. Although discharge of polluted effluents from combined overflows and storm sewers during a storm are definitely a dynamic process, a number of assumptions were made to reduce the system to one suitable for a steady state analysis by means of the Streeter-Phelps equation. These are given in Table 3.

The resulting wet weather system is shown schematically in Figure 6a.

Average wet weather inputs from the combined and storm sewered areas were determined from the output of the STORM model. The joint use of STORM and the Streeter-Phelps equation enabled several years of record to be considered at a minimal computer cost.

In order to facilitate the analysis of a large number of storm events, a series of generalized curves were developed. For given initial conditions, these were used in conjunction with the simulated pollutant inputs for each event from STORM to estimate the maximum dissolved oxygen deficit. Figure 8 presents the deficit curves applicable to events for which the total stream flow (baseflow + average stormflow) results in a velocity of less than or equal to 1.56 m^3/s. Various other sets of curves corresponding to higher flow ranges (shorter retention times) were also developed. For demonstration purposes, the SWMM RECEIV Block was also used to simulate a selected storm event in the Creek and to confirm this simplified approach.

The evaluation of receiving water impacts was based on the MOE minimum criterion of 4 mg/l for dissolved oxygen levels in surface waters. The average concentrations of TOD for each dry day and for each storm runoff event were input to the Streeter-Phelps DO curves to determine if this criterion would be violated. The relative importance of wet and dry weather conditions was then determined on the basis of the total number of violations incurred for each over the seven year period analysed. It was found that the wet and dry weather conditions were equally severe as both resulted in a large number of violations annually, as shown in Tables 4 and 5.

Evaluation of Alternatives

In evaluating alternatives, the emphasis was placed on pollutant loading to Kettle Creek, rather than on the receiving water response. Pollution abatement measures were accordingly evaluated for their potential to reduce the BOD loadings as an index of both receiving water response and total loading to the Creek. After screening alternatives using a scoring matrix, the highest ranking alternatives were then analysed using the STORM model. Wet weather pollution abatement alternatives considered were:

1. street sweeping
2. sewer separation
3. in-system storage
4. off-line storage and treatment

Dry weather alternatives were:

1. tertiary treatment
2. pipeline to Lake Erie
3. low flow augmentation

Figure 9 shows the relative effectiveness of various alternatives based on the STORM model results. Cost estimates were also developed for these alternatives and a cost effectiveness curve was then prepared (Figure 10). A similar cost-effectiveness curve for dry-weather abatement alternatives is shown in Figure 11.

These curves, coupled with subjective matrix scores, were used for screening pollution abatement alternatives. The results for water quality alternatives were then combined with the results of a similar screening process for relief alternatives to arrive at an integrated management scheme, as shown in Figure 12.

CONCLUSION

The results of this study indicate the ability of relatively simplified modelling techniques to account for the system interactions in urban drainage applications for planning purposes. Various methods were developed which enabled a large number of alternatives to be considered for both flood control and pollution abatement utilizing a judicious interfacing of computerized models and manual techniques. Furthermore, the methods developed are generalized and applicable to any similar planning application. As a result of this process, it was possible to obtain a clear indication of where the limited financial resources of the City should be invested for maximum benefit.

TABLE 1

RELIEF ALTERNATIVES TESTED ON TYPICAL TEST AREAS

ALTERNATIVE	TEST AREAS RES = RESIDENTIAL COMM = COMMERCIAL	RETAIN EXISTING COMBINED LATERAL	RUNOFF REDUCTION (DISCONNECT ROOF[1]/ ROOF, PARK.LOT STOR.[2])	3",6",9" STREET PONDING	IN-SYSTEM STORAGE ELEMENT	DESIGN LEVELS (YRS)	1 = NEW COMBINED LATERAL 2 = NEW SEPARATED LATERAL
1	Res & Comm.					2-25	1
2	"	x	x		x	"	
3	"	x	x	x	x	"	
4	"	x	x	x		"	
5	"	x			x	"	
6	"	x				"	2
7	"	x	x			"	2
8	"	x	x	x		"	2
9	"	x	x	x	x	"	2
10	"	x	x		x	"	2
11	Res -LU Existing					"	2
12	"		x			"	2
13	Redevelopment					"	2
14	"		x			"	2

1. in Residential areas only

2. in Industrial/Commercial areas only

TABLE 2

ST. THOMAS - MEASURED

AVERAGE EFFLUENT CONCENTRATIONS (mg/l)

(initially assumed values in brackets)

	TOTAL SS	BOD_5	FREE AMMONIA as N	TOTAL P
Combined Overflows	321 (222)	100 (40)	1.5*	1.5*
Separated Runoff	272 (200)	23 (13)	0.5*	0.5*
Secondary Treated Effluent	17.4	17.4	2.3	0.9

* From Literature

SS = Suspended Solids

P = Phosphorus as P

BOD_5 = 5-day Biochemical Oxygen Demand

TABLE 3

SUMMARY OF ASSUMPTIONS FOR WET WEATHER DO ANALYSIS

ASSUMPTIONS

1. Basic Streeter-Phelps Assumptions
2. Upstream BOD = 7.3 mg/l
3. Kd = 1.6; Ka = 0.8
4. Upstream DO Deficit = 0
5. Travel Time vs Flow rate from WSC Gauge Station
6. DO less than 4 mg/l is the Violation Criterion
7. Upstream and Downstream Runoff independent of St. Thomas Runoff
8. Base Streamflow on day of rain is equal to average flow from preceeding day
9. Treatment Plant effluent = .18 m^3/s
10. Concentrations as per Table 1

FACTORS ACCOUNTED FOR

- Flow travel time to Lake Erie
- NOD
- Upstream BOD
- Dissolved Oxygen saturation proportional to Temperature
- Treatment Plant effluent

FACTORS NOT ACCOUNTED FOR

- Benthic BOD
- Diurnal Effects
- Variable K values
- Upstream and Downstream Runoff
- Upstream and Downstream DO Deficit

TABLE 4

KETTLE CREEK - FREQUENCY OF DO VIOLATIONS (DAYS) 1968-1974 DURING DRY WEATHER*

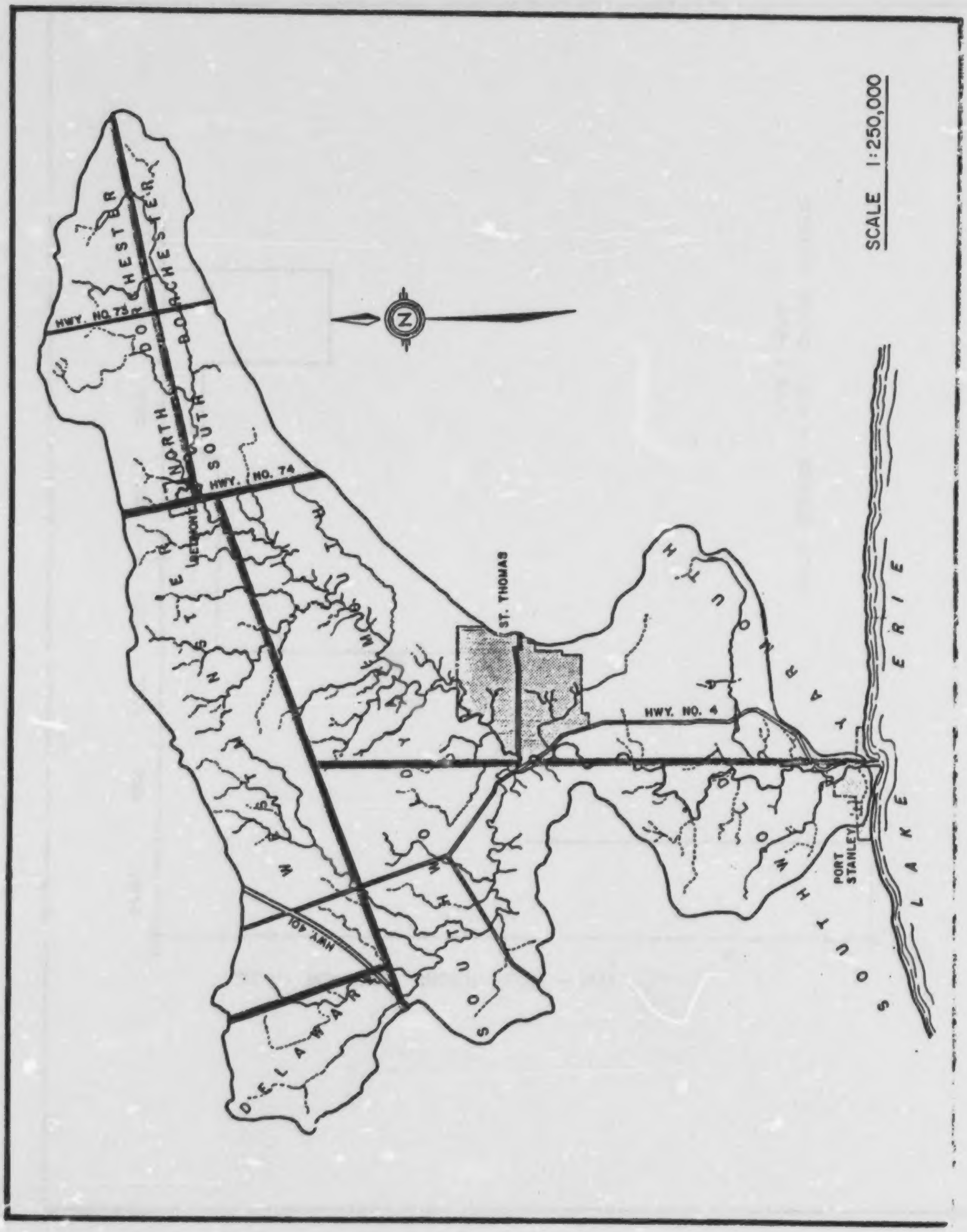
YEAR	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	TOTAL
1968	0	0	2	5	7	2	0	0	16
1969	0	0	0	7	26	30	18	0	81
1970	0	0	9	16	30	27	8	0	81
1971	0	0	16	26	23	27	29	15	135
1972	0	0	1	0	0	0	0	0	1
1973	0	0	0	10	12	26	18	0	66
1974	0	0	0	2	0	3	0	0	5
TOTAL	0	0	28	66	98	115	73	15	395
AVERAGE	0	0	4.0	9.4	14.0	16.4	10.4	2.1	56.4
TOTAL # OF DRY DAYS	182	167	168	182	191	193	173	193	1449
DAYS VIOL./ DRY DAYS	0	0	17	36	51	60	42	8	27
% OF ALL VIOLATIONS	0	0	7.1	16.7	24.8	29.1	18.5	3.8	100

*Days with less than 2.5 mm rainfall and less than 1.2 mm in any hour.

TABLE 5

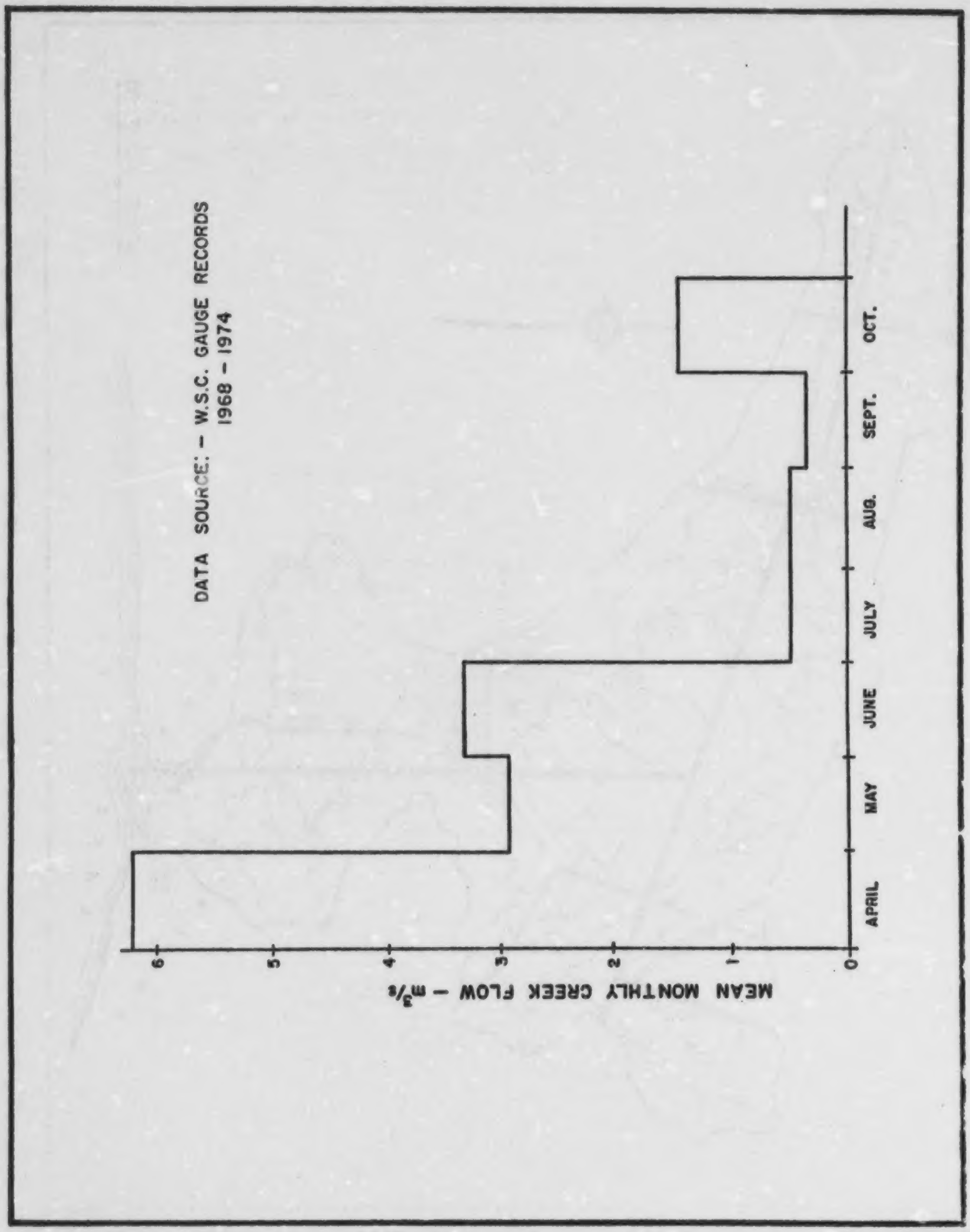
WET WEATHER DO DEFICIT SUMMARY
1968-1974

	DISSOLVED OXYGEN DEFICIT VALUES mg/ℓ							VIOLATIONS IN MONTH AS	
MONTH	0-2	2-4	4-6	6-8	8+	VIOLATIONS	EVENTS	% OF DAYS	% OF ALL VIOLATION
April	0	4	1	3	19	20	27	9.5	9.7
May	0	2	7	3	33	35	45	16.1	16.9
June	2	5	5	3	31	36	46	17.1	17.4
July	0	0	0	0	32	29	32	13.4	14.0
August	1	0	0	2	24	24	27	11.1	11.6
September	0	0	2	0	27	28	29	13.3	13.5
October	0	1	1	3	36	35	41	16.1	16.9
TOTAL EVENTS	3	12	16	14	202	207	247	13.8	100
AVERAGE	0.4	1.7	2.3	2.0	28.9	-	-	-	-


HWY. NO. 73
NORTH DORCHESTER
SOUTH DORCHESTER
HWY. NO. 74
BELMONT
WESTMINSTER
ST. THOMAS
HWY. NO. 4
YARMOUTH
SOUTHWOLD
HWY. 401
DELAWARE
PORT STANLEY
LAKE ERIE
SOUTH YARMOUTH
N
SCALE 1:250,000

Kettle Creek Drainage Area

FIGURE 1

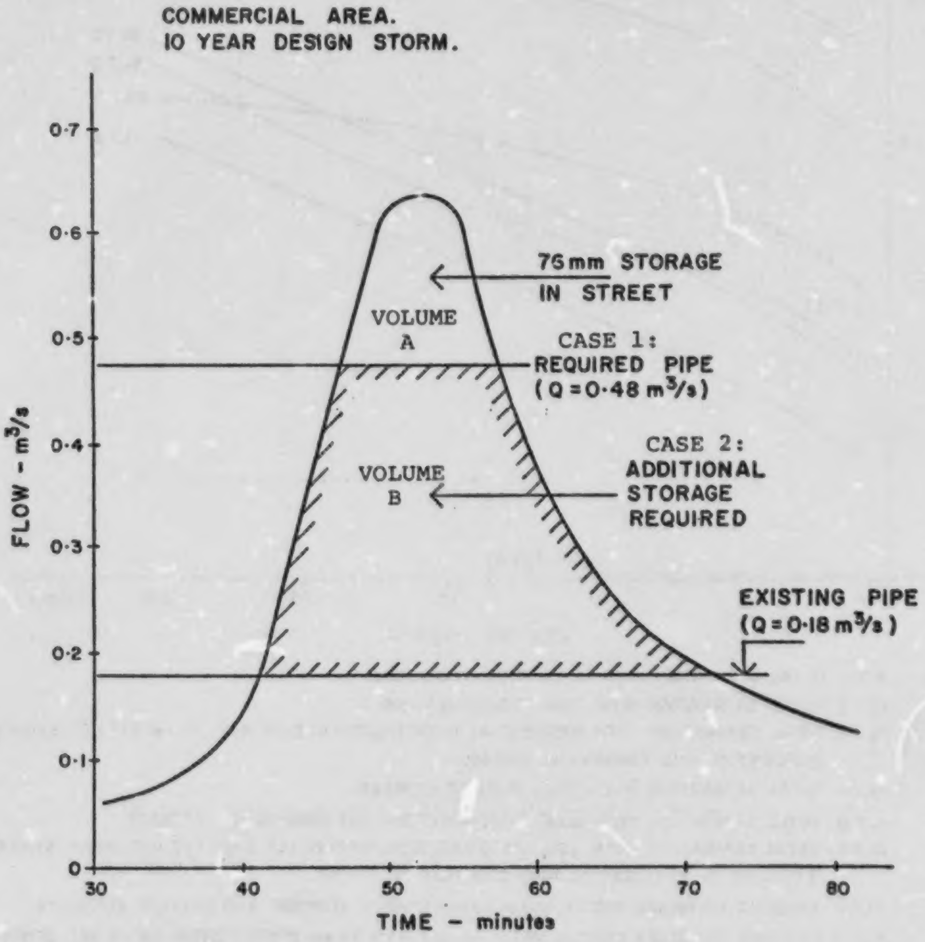


Mean Monthly Flows
In Kettle Creek

FIGURE 2

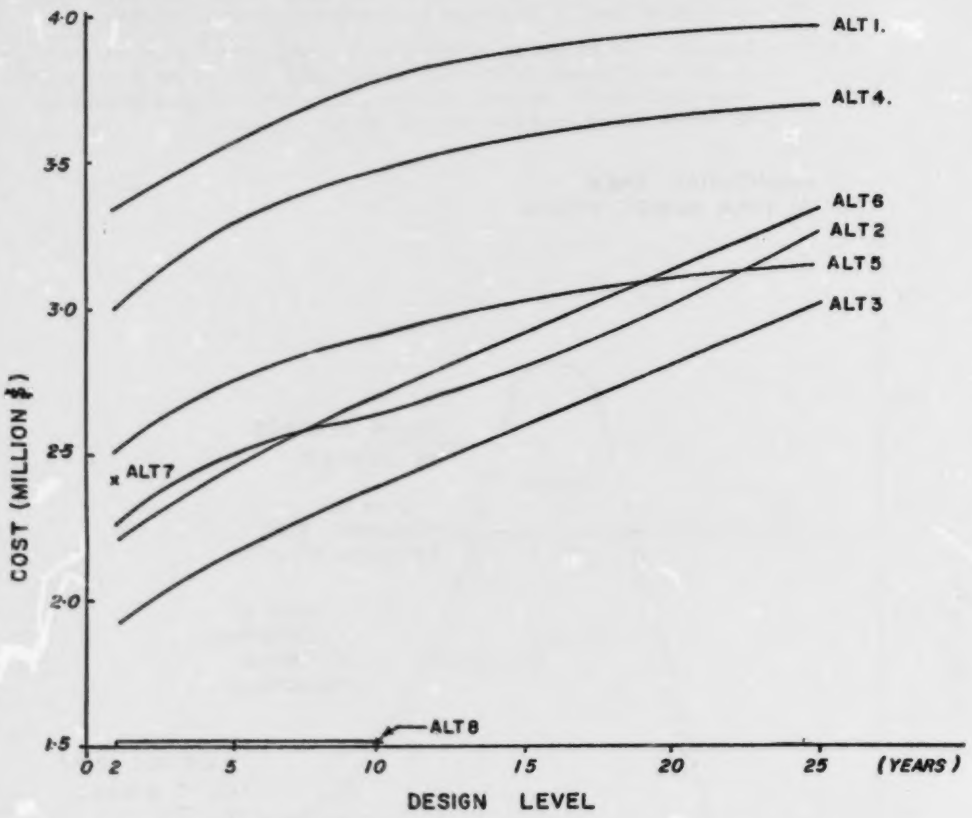
EXAMPLES: Case 1 - Permissible depth in street of 76 mm resulting in storage volume A. Using this storage a sewer capacity of 0.48 m^3/sec is required to prevent basement flooding.

Case 2 - Assuming the existing sewer with a capacity of 0.18 m^3/sec is to be relieved by storage alone, and only 76 mm is permitted on the street (Volume A). An additional volume of underground storage (Volume B) is required.



Example Of Procedure For Evaluation Of Relief Alternatives

FIGURE 3



ALT.1. TOTAL SEPARATION WITH NO CONTROL MEASURES.

ALT.2. TOTAL SEPARATION WITH ROOF DISCONNECTION

ALT.3. TOTAL SEPARATION WITH RESIDENTIAL ROOF DISCONNECTION AND 75mm STREET STORAGE IN RESIDENTIAL AND COMMERCIAL AREAS.

ALT.4. TOTAL SEPARATION WITH 75mm STREET STORAGE.

ALT.5. TOTAL SEPARATION WITH ROOF DISCONNECTION AND PARKING LOT STORAGE.

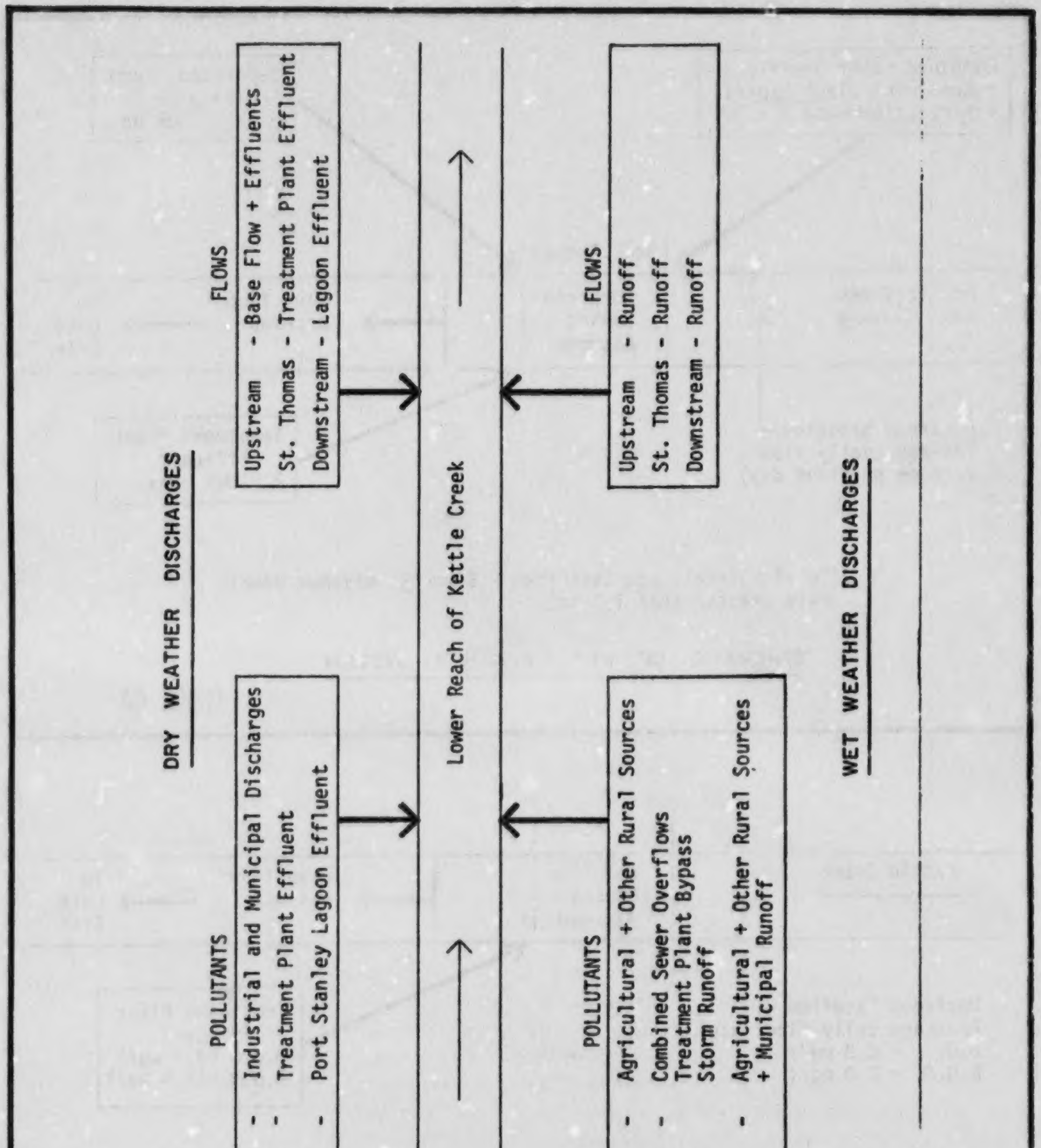
ALT.6. TOTAL SEPARATION, WITH ROOF DISCONNECTION, PARKING LOT STORAGE AND 75mm STREET STORAGE IN RESIDENTIAL AND COMMERCIAL AREAS.

ALT.7. EXISTING COMBINED SYSTEM WITH 75mm STREET STORAGE AND STORAGE ELEMENTS.

ALT.8. EXISTING COMBINED SYSTEM WITH 150mm AND 75mm STREET STORAGE IN RESIDENTIAL AND COMMERCIAL AREAS, RESPECTIVELY.

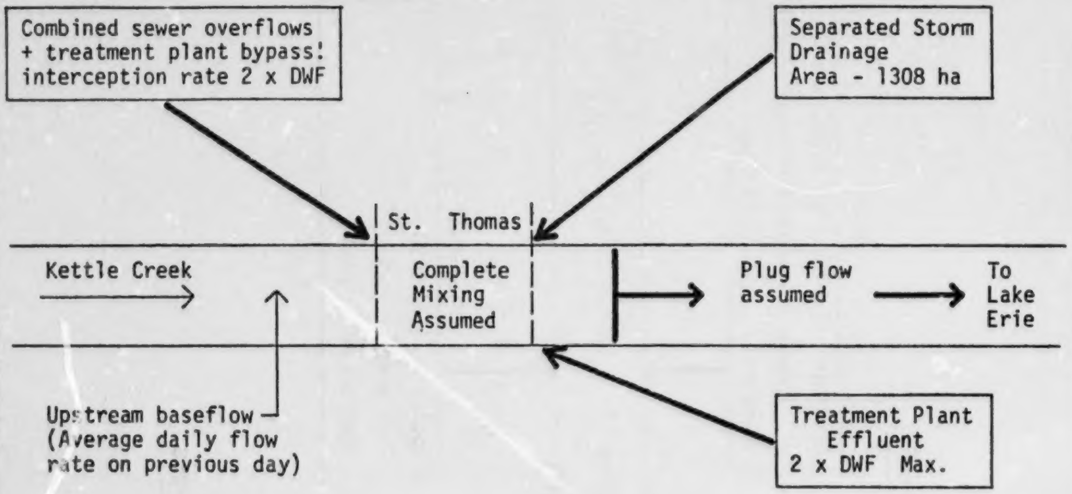
Area D-2 Cost Estimates
For Relief Alternatives

FIGURE 4


DRY WEATHER DISCHARGES
POLLUTANTS
Upstream - Industrial and Municipal Discharges
St. Thomas - Treatment Plant Effluent
Downstream - Port Stanley Lagoon Effluent
FLOWS
Upstream - Base Flow + Effluents
St. Thomas - Treatment Plant Effluent
Downstream - Lagoon Effluent
Lower Reach of Kettle Creek
POLLUTANTS
Upstream - Agricultural + Other Rural Sources
St. Thomas - Combined Sewer Overflows
Treatment Plant Bypass
Storm Runoff
Downstream - Agricultural + Other Rural Sources
+ Municipal Runoff
FLOWS
Upstream - Runoff
St. Thomas - Runoff
Downstream - Runoff
WET WEATHER DISCHARGES

Elements Of The Kettle Creek Water Quality System

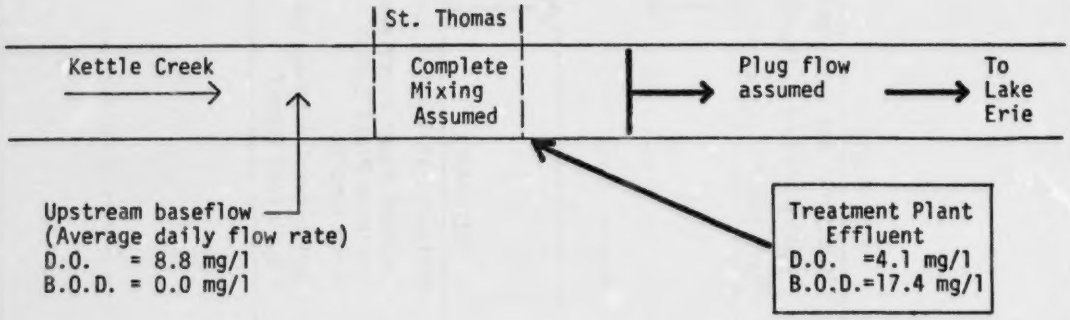
FIGURE 5


Combined sewer overflows
+ treatment plant bypass:
interception rate 2 x DWF
Separated Storm
Drainage
Area - 1308 ha
St. Thomas
Kettle Creek
Complete
Mixing
Assumed
Plug flow
assumed
To
Lake
Erie
Upstream baseflow
(Average daily flow
rate on previous day)
Treatment Plant
Effluent
2 x DWF Max.

(Total rainfall not less than 2.5 mm OR maximum hourly rate greater than 1.3 mm)

SCHEMATIC OF WET - WEATHER SYSTEM

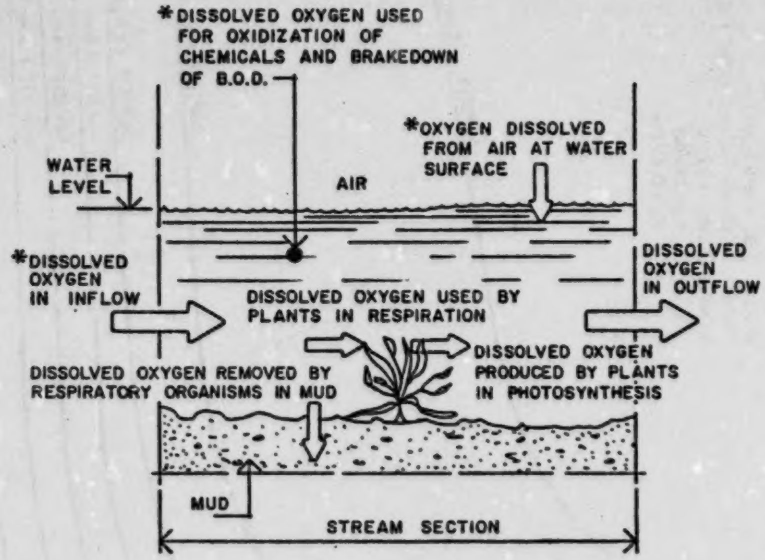
FIGURE 6A


St. Thomas
Kettle Creek
Complete
Mixing
Assumed
Plug flow
assumed
To
Lake
Erie
Upstream baseflow
(Average daily flow rate)
D.O. = 8.8 mg/l
B.O.D. = 0.0 mg/l
Treatment Plant
Effluent
D.O. =4.1 mg/l
B.O.D.=17.4 mg/l

SCHEMATIC OF DRY - WEATHER SYSTEM

FIGURE 6B

Schematic Of Loadings From St. Thomas


*DISSOLVED OXYGEN USED FOR OXIDIZATION OF CHEMICALS AND BRAKEDOWN OF B.O.D.
*OXYGEN DISSOLVED FROM AIR AT WATER SURFACE
WATER LEVEL
AIR
*DISSOLVED OXYGEN IN INFLOW
DISSOLVED OXYGEN USED BY PLANTS IN RESPIRATION
DISSOLVED OXYGEN IN OUTFLOW
DISSOLVED OXYGEN REMOVED BY RESPIRATORY ORGANISMS IN MUD
DISSOLVED OXYGEN PRODUCED BY PLANTS IN PHOTOSYNTHESIS
MUD
STREAM SECTION

STREETER-PHELPS EQUATION*

$$\text{D.O. deficit} = \frac{K_d L_a}{K_a - K_d}\left[e^{-K_d t} - e^{-K_a t}\right] + D_a e^{-K_a t}$$

where: K_d = deoxygenation rate constant

K_a = re-oxygenation rate constant

L_a = BOD ultimate in inflow to system (mg/l)

D_a = D.O. deficit in inflow to system (mg/l)

N.B. D_a = saturation concentration-actual concentration

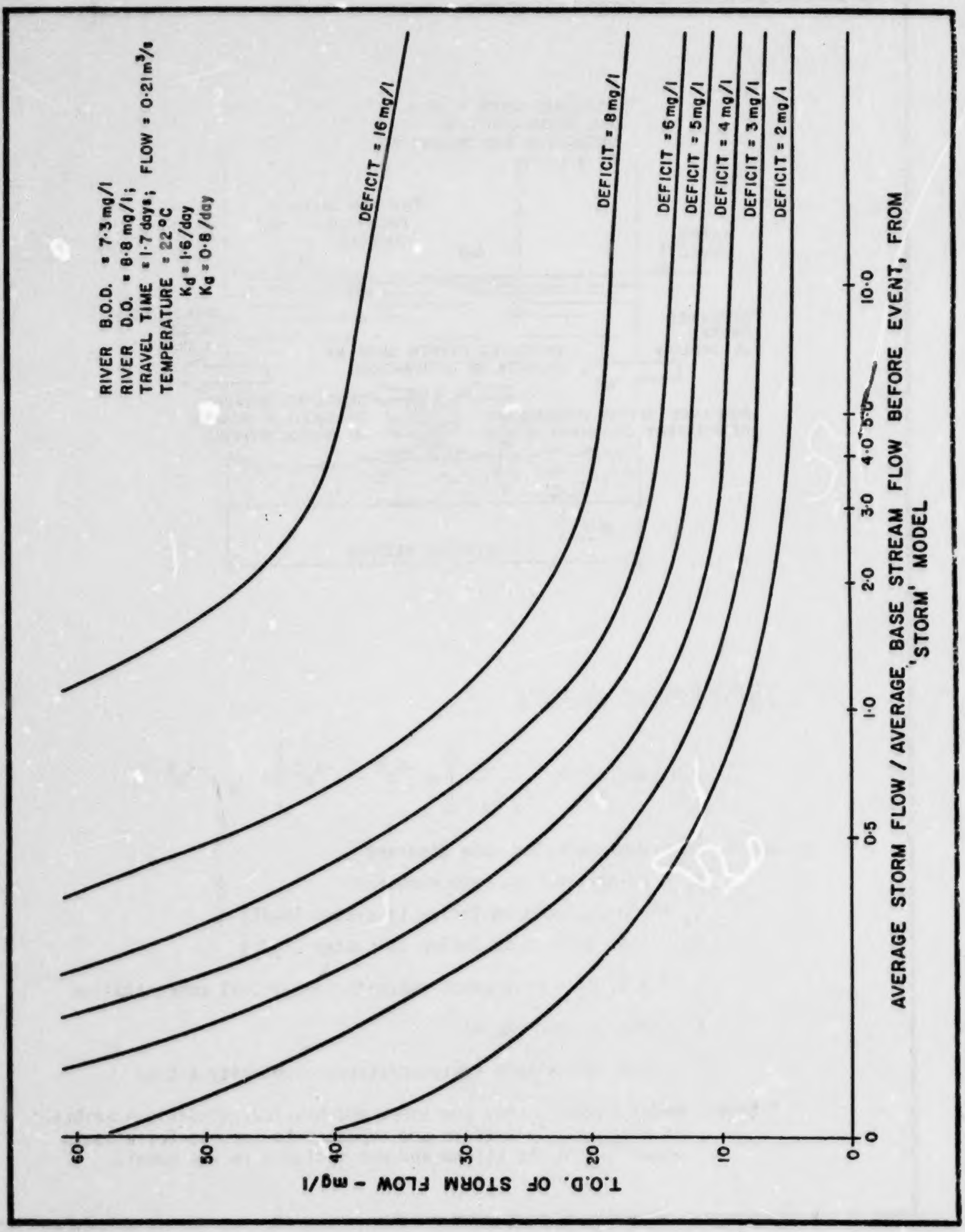
t = travel time (days)

N.B. for simple system distance = velocity x time

* Note: model includes only processes and boundary conditions marked * in diagram. Parameters are adjusted to account for effects occurring in the stream and not included in the model.

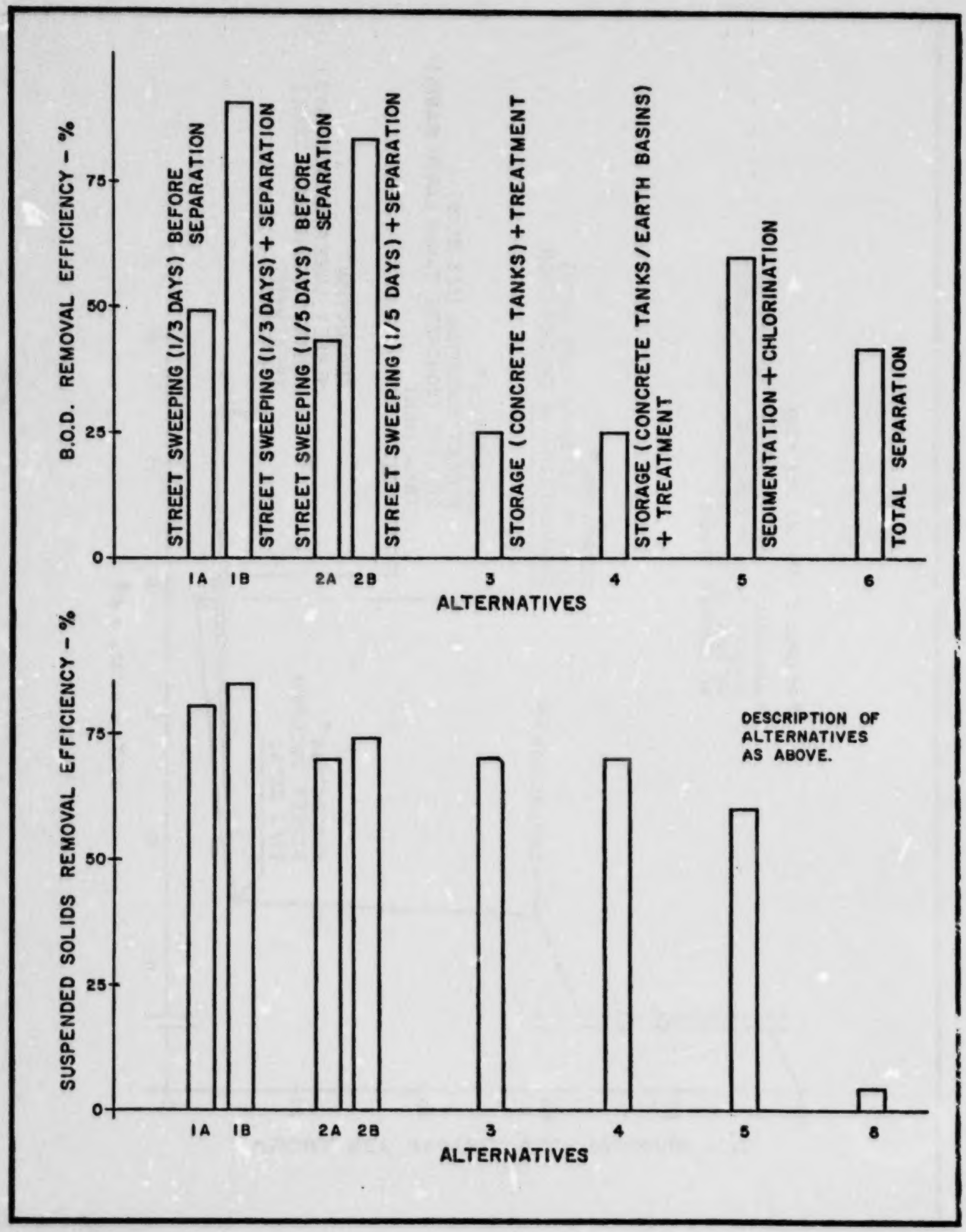
Oxygen Balance And The Streeter-Phelps Equation

FIGURE 7



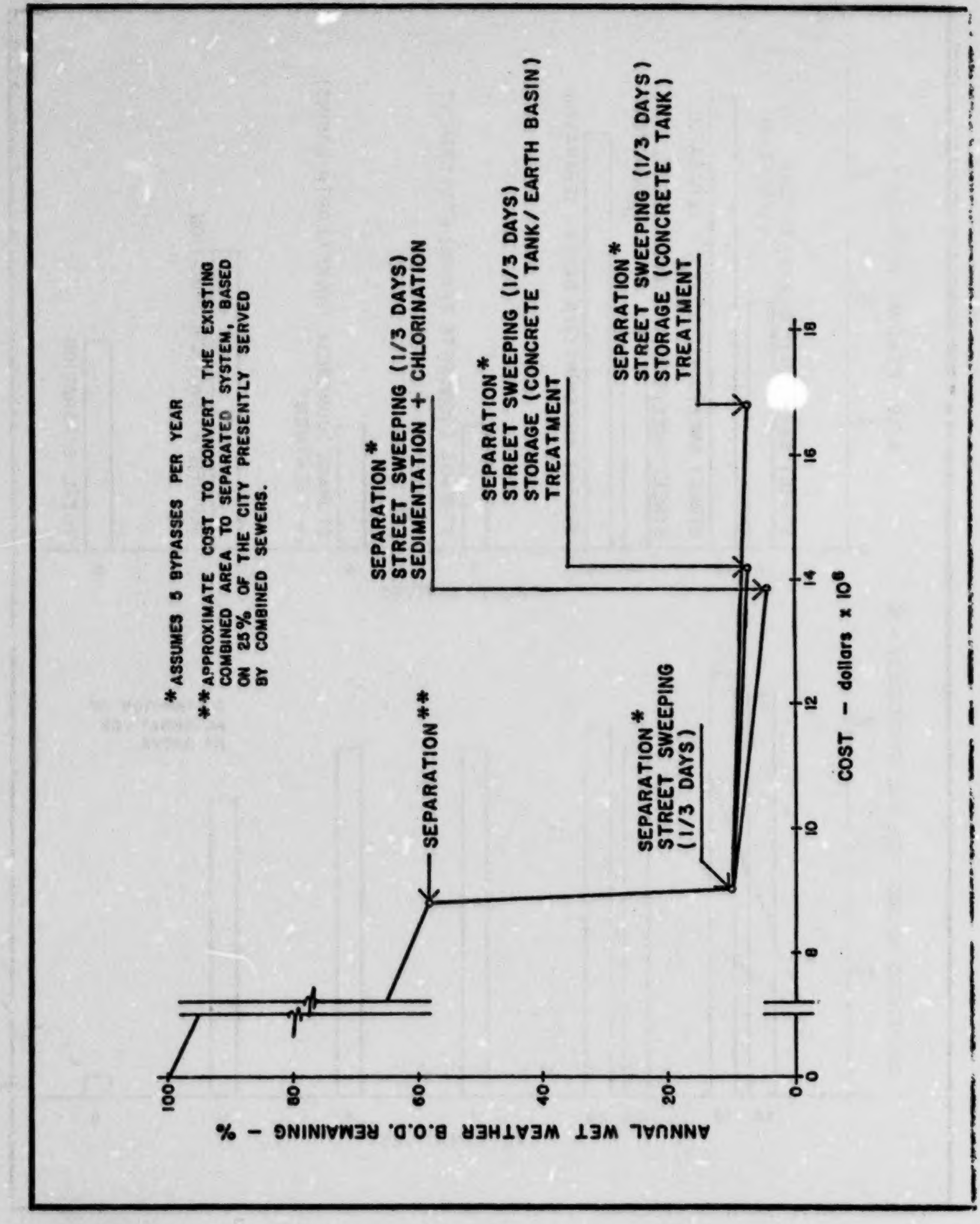
Dissolved Oxygen Deficits
For Various Conditions

FIGURE 8



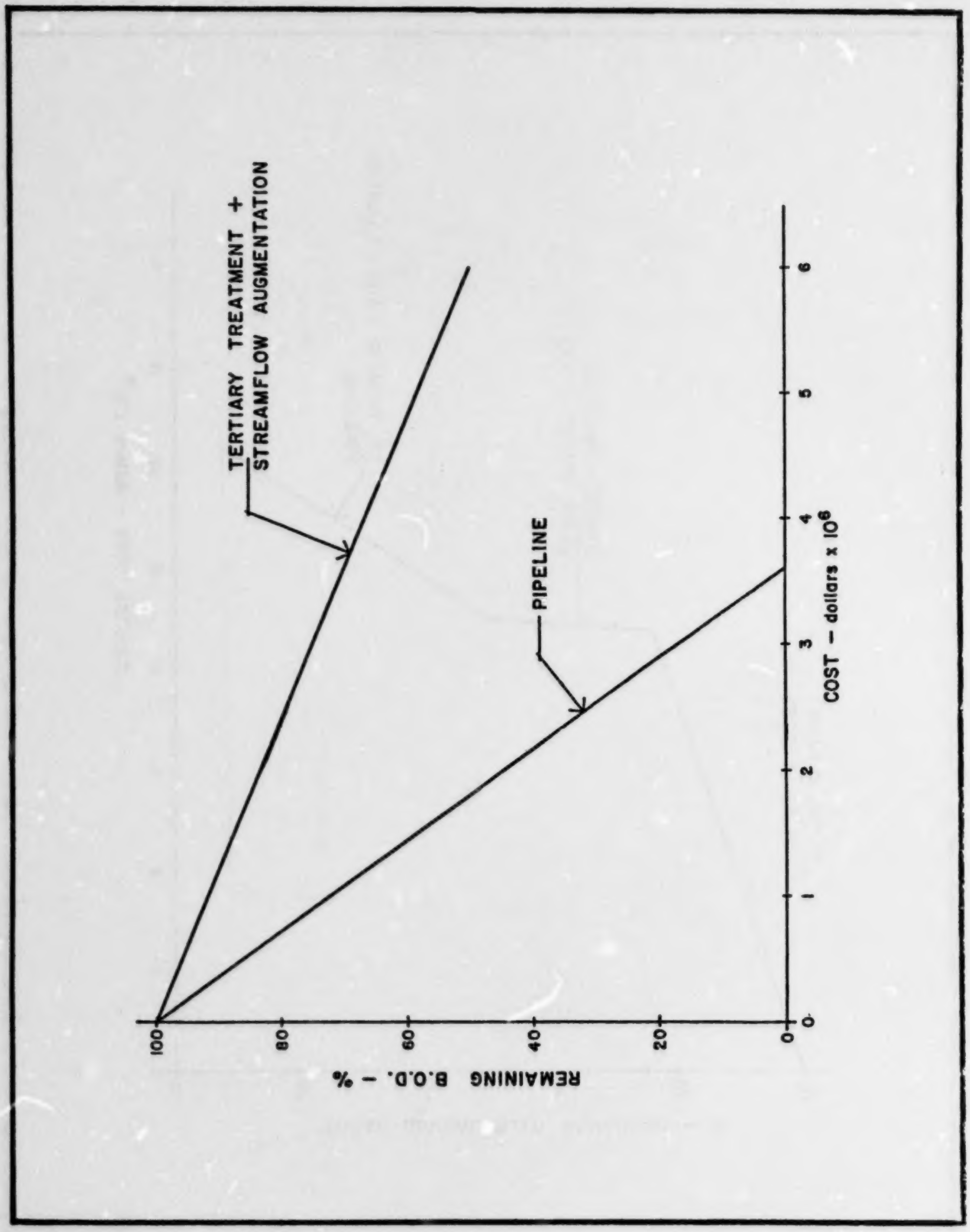
Effect Of Wet Weather Pollution Abatement Options

FIGURE 9



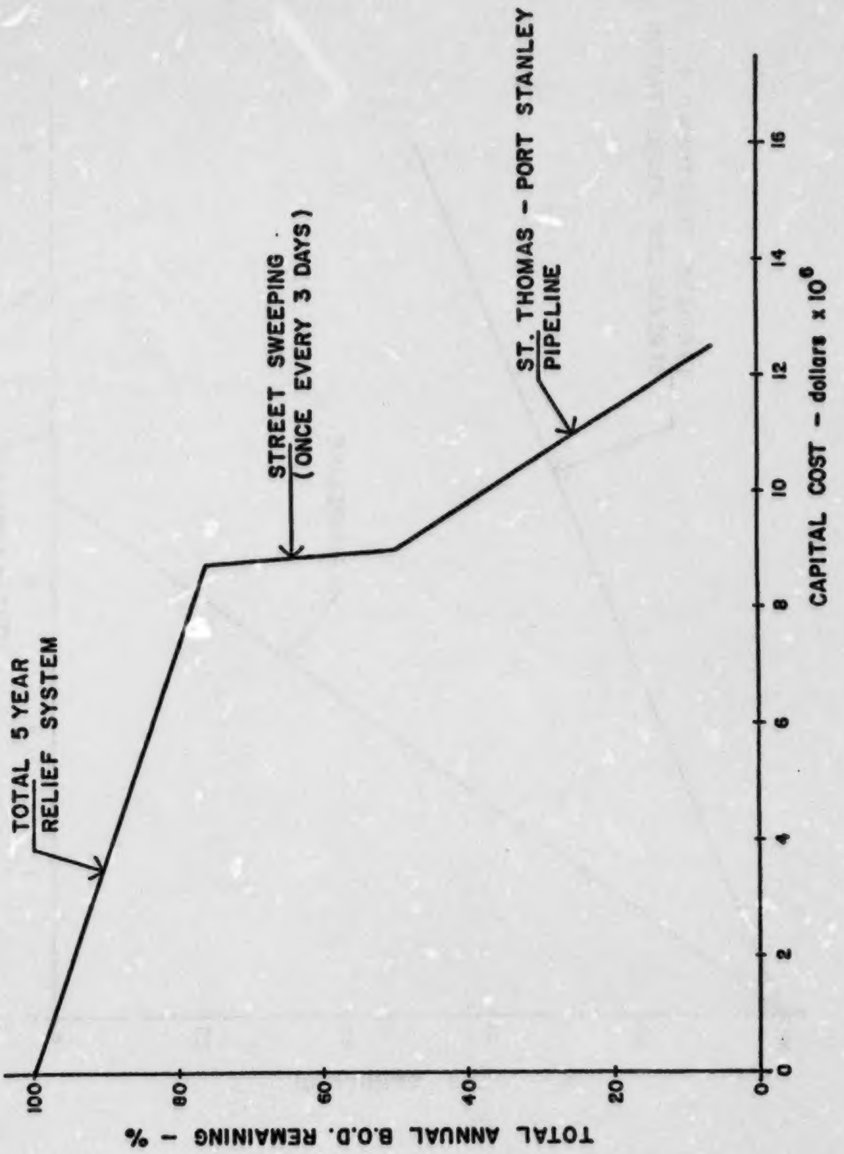
Cost Effectiveness Of Wet Weather Pollution Abatement

FIGURE 10



Cost Effectiveness Of Dry
Weather Pollution Abatement

FIGURE 11



Cost Effectiveness Of Best Alternatives

FIGURE 12

SWMM USERS GROUP MEETING
OTTAWA ONTARIO - 4-5 MAY 1978

APPLICATION OF SWMM - EXTRAN FOR THE ANALYSIS OF EXISTING URBAN DRAINAGE SYSTEMS

BY

CHRISTIAN W. EICHER, GORE & STORRIE LIMITED
CONSULTING ENGINEERS, TORONTO, ONTARIO

INTRODUCTION

This paper is a collection of three sample applications of the Extended Transport Block of the EPA - Storm Water Management Model. The three examples describe situations in existing systems which cover a diverse spectrum of problems and solutions.

EXAMPLE NO. 1 Borough of East York - Development of Relief Solutions for Central Area Sewer System

Development of interim relief solutions for a large combined sewer system with extensive and frequent basement flooding on the basis of an ongoing long-term separation programme.

EXAMPLE NO. 2 Borough of Scarborough - Evaluation of Major/Minor Drainage Performance of a Flood Relief Scheme for an Existing Residential Subdivision

Comparative performance analysis of two basement flooding relief concepts for a relatively new subdivision with separate sewers.

EXAMPLE NO. 3 Borough of Scarborough - Analysis of Impact of Urbanization on Bendale Creek Drainage Channel

Continuous backwater simulation for a large open drainage channel and the tributary storm sewer systems to assess the relative impact of upstream urbanization and the implementation of runoff control measures (zero-increase-in-runoff concepts).

Apart from the description of the area and the problems that led to the application of the simulation model, comments are made regarding several obstacles encountered in the course of the use of the programmes, and suggestions are given for alternatives as well as further improvements to the Extended Transport Block.

The usefulness of EXTRAN in the realistic simulation of extremely complex hydraulic phenomena has been clearly demonstrated in the presented examples, as well as in other small-scale applications. The difficulties encountered are certainly quite often frustrating and over-time consuming.

However, they also reflect the challenge to attach more and more complex problems as a consequence of having a very powerful engineering tool at hand. The SWMM Model with the Extended Transport Block definitely ranks among the best techniques available to the general user at the present time.

BOROUGH OF EAST YORK - Development of Relief Solutions for Central Area Sewer System

The sewer system in the Central Area of East York is of the combined type, as in many of the older parts of Metropolitan Toronto. These systems have been built to generally much lower design standards than are in use presently. They also suffer from their inherent problematic of having the sanitary sewage, the stormwater from the roofs and groundwater from the foundation drains, all connected to one conduit, as well as the connection from the basement floor drain. Widespread basement flooding has been almost a fact of life for many homeowners in critical locations. In addition, redevelopment and more extensive land use have increased the percentage of impervious area steadily over the years, thereby worsening the surcharge situation noticeably.

East York's Central Area is among the hardest hit districts in Toronto in terms of basement flooding. The Municipality has recognized the need for relief already years ago, and several studies were commissioned in order to determine the most suitable methods for relief. It is obvious that the general name of the game has been separation of the sewer system, with the existing combined system handling the sanitary flow and the runoff where roofleader disconnection was not an obvious and logical solution.

With the available outlets rather limited and with street alignments and the political boundaries not normally in line with watershed limits, the proposed storm sewer schemes became very long and expensive. This, together with a number of dry summers, led to slow progress on the implementation of the proposed separation programmes.

In 1975, Gore & Storrie have been asked by the Borough to re-assess the validity of the reports from 1963, after several important segments of the separation had been completed, with reasonable improvement of the flooding situation, but also at substantial cost. A further part of the Terms of Reference was to look into the possibility of interim solutions which would provide some early benefit from the big trunk tunnels, forming the backbone of the new storm sewer system. The report confirms the validity of the general storm sewer scheme and suggests a number of interim solutions. One of them is the evaluation of selected relief measures arrived at, alleviating the overload in the existing combined sewer system in strategic locations. This evaluation is based on a thorough analysis of the extremely complex sewer system by means of the SWMM Model with the Extended Transport Routine. It includes an assessment of the improvements obtained with relief overflows at key locations in the system as well as local reinforcements of bottlenecks.

The Central Area combined sewer system has its outlet north to a large overflow chamber, located at the Don Valley Parkway. The stormwater overflows the Don River, and the dry weather flow is intercepted to the Metro Interceptor system. The overflow chamber represents what used to be the state-of-the-art and certainly provides little protection for the Don River.

The entire drainage system is far too big for detailed analysis, and had to be done in several segments. However, for the development of the boundary conditions, i.e. the water levels in the trunk sewers and the assessment of the capacity and performance of the overflow with the outlet sewer to the Don River, the entire system had to be simplified and modelled in a preliminary phase. Despite the location of the overflow chamber high above the Don River, the water level in the outfall rises to within a few feet of the overflow. This is using the simulation of an actually measured storm as the model input.

The surcharge water levels developed in the simulation of the entire drainage area have been used subsequently as the basis for the detailed analysis of different segments of the system. The surcharge levels have been compared with observations made by the Works Department, as well as some results from a monitoring study carried out on part of the watershed by M.M. Dillon Limited.

In the simulations, water in the sewer system was allowed to raise to street level, from where it was routed to other inlets, overflowing to other drainage watersheds or simply ponding until it would re-enter the system. This required the addition of surface channels and storage elements in the model. This was found necessary because otherwise water would be lost from the system unaccounted, thus reducing the effective load on the system.

The improvement resulting by a single relief measure or a set of modifications is clearly shown by the reduced levels of surcharge in the system. The output from the Extended Transport Routine indicates also how far-reaching the effects of relief overflows for the trunk system are and where local storm sewers would still be required. This is generally the case in the upstream branches of the system where even a free discharge on a typical 12-inch sewer could not prevent this from surcharging, and local relief is required.

Significant findings from the study as far as it has been completed to date are:

1) The locally-looped and interconnected systems in some parts of East York have markedly better performance than single-ended branches, because of the possible interaction making use of the dynamic character of the runoff.

2) The trunk and collector system is in many areas the main cause of the surcharge problems, more so than the local sewer systems. This is somewhat contrary to earlier findings based on Rational - Formula analysis which indicated that the trunk system would have comparably better capacity. This became clear when the timing of the peaking of the surcharge levels was analyzed. It was also confirmed by information obtained from house-to-house surveys. The reason for this discrepancy is most likely the result of the generally simplified procedures used in the normal application of the Rational Formula, particularly in the computation of flow at major junctions, which has been found to lead to the underestimation of the actual load.

3) Relieving the existing combined sewer system at key locations can significantly improve the protection provided by the existing system without increasing the number of overflow events to any great extent. The overflow weirs are designed with crest elevations at the obvert of the combined sewer and without restriction of the downstream flow. They act, therefore, only as surcharge relief during a limited number of storm events per year.

4) The proposed layout of storm separation schemes can be optimized in many cases such that the most needy areas can be relieved without immediately implementing the entire scheme. This can provide considerable improvement in a shorter time than would be possible if the separation programme had to be based on the ultimate scheme. Some schemes that provide early relief for the most flooded areas may not appear optimized for the ultimate system. Their overall effectiveness is substantially better, however, because they provide for the allocation of funds for relief projects in other areas of the Borough at an earlier date. It is obvious, however, that the requirements of the ultimate separation scheme have to be taken into full account.

The water quality concern: It is quite clear that the proposed relief by means of additional overflows from the combined sewer system as an interim measure may raise concern regarding the impact on the quality of the receiving waters. The Municipality is fully aware of the problem, but it must be recognized that with the limited funds available for sewer system improvements, the priority is clearly in the protection of homes against the hazards and the inconvenience of basement flooding.

The water quality aspect is taken into consideration as far as possible in the design of the relief structures, as well as in the layout of the storm sewer outfalls. Overflow weirs are built up to the obvert of the combined trunk sewer which will, therefore, operate at its maximum capacity before any overflow occurs. Storm sewer outfalls are designed such that eventual future storage or treatment facilities can be accommodated without costly complications such as pumping.

In addition, it must be realized that the surcharge relief overflows installed at the combined sewer system are strictly meant to be interim measures. With the gradual development of the entire ultimate storm sewer scheme, the combined sewers will hopefully cease to surcharge. This would then also make it possible to improve the existing overflow structure at the outfall of the combined system in order to provide a better protection of the receiving stream. The present surcharge of the combined system would make any modifications of the simple interceptor chamber virtually impossible.

BOROUGH OF SCARBOROUGH - Evaluation of Major/Minor Drainage Performance of a Flood Relief Scheme for an Existing Residential Subdivision

Several areas in the Borough of Scarborough have a substantial record of basement flooding and ponding of water on streets, parks and parking lots.

One area, known as the Subtrunk "D" Watershed, has a particularly bad history of problems, starting right from the beginning when the first subdivisions were developed in the late fifties and early sixties. The area has separate sewers. The storm sewers have been designed for a five-year recurrence storm with runoff coefficients of 35 and 45 percent. It would seem that the lower coefficient was in force when the trunk system was designed, and the higher value was used for the local sewers. This indicates already one reason for the flooding problems, caused primarily by overloaded storm sewers. A secondary reason is the connection of the foundation drains to the storm sewer which allows the rising water levels in the system to exert pressure onto basement floors and walls with resulting leakage or break-up.

The proposed method for flood relief for the entire area is based on a storage scheme, together with the segmentation of the watershed into isolated sections, with the trunk sewers in each segment capable of handling the runoff from a major storm. This would at the same time provide for improved outlet conditions for the local branch sewers. The comparably high protection proposed for the trunk sewers has been justified by the inability of the system to sustain even minor surcharges, as well as the fact that overland flow on the streets along the trunk sewers would result in intolerable depths of flow and ponding. The subdivision design in the Subtrunk "D" watershed has not provided for flow routes and outlets for overland flow, although some parts of the area would allow limited application of this concept. Basement elevations are as low as only 2 to 3 feet above the storm sewer obvert in many areas.

The concept of breaking the existing system into a number of largely independent subsystems has emerged out of the initial runoff simulations undertaken to assess the magnitude of inadequacy of the existing conduit system. This has been carried out, using the Runoff and Normal Transport Blocks of the SWMM Model. Using the re-design option of the Transport Block, it was possible to obtain a rough estimation of what the actual runoff and the required conduit sizes would be for different design storms. Similarly, the system performance has been assessed assuming different combinations of subsystems, depending on where potential storage facilities were available to retain the flow from upstream.

A more detailed analysis has been subsequently undertaken for the upstream section of the watershed, known as the Bridlewood Subdivision. This analysis served to compare the performance of a system with overland flow routing for major storm runoff against the concept of providing increased trunk sewer capacity by means of breaking the system into more or less independent segments with storage of the upstream flows. The requirements of modelling both conduit and overland flow, as well as the full consideration of dynamic backwater conditions have led to the application of the Extended Transport Block, together with the Runoff Block of the SWMM Model.

The combined operation of the surface and conduit systems has been represented in the following manner:

- The storm sewer system has been simulated in the usual way such that it would represent as close as possible the sizes and slopes of the existing conduits. Some adjustment was necessary in the location of the manholes in order to have them coincide as far as possible with the inlets at the low points in the street grade.

- The surface system has been represented by a system of trapezoidal channels with five percent side slope. The trapezoidal profile had been abstracted from the two roughly triangular channels of the roadway with the curb and the extension of the flow profile across the boulevard and sidewalk. The side slope represents the average shape of the roadway profile and has been determined from contour plans and field survey.

- The connections between the surface channels and the conduit system has been represented by orifices which were calibrated to represent average catchbasin capacities or, in some cases, the inlet restrictions required to prevent the sewer system from surcharging.

- The roof areas, with some allowance for driveway catchbasins, have been simulated to be connected to the conduit system, except where a proportion of the roofs could be assumed to discharge onto the ground.

- The remaining areas have been assumed to connect to the surface system. Backyard catchbasins have not been included, although they connect to the conduit system. Their capacities, however, are small relative to the street sewer.

- Interceptor sewers to storage, as well as the storage facilities themselves, have been modelled according to the assumptions of the original concept. Some adjustments have been made in the course of the analysis to optimize the design in order to compensate for the lack of detail in the initial analysis.

During this application of the Extended Transport Block as well as earlier small-scale test applications, it became evident that a number of modifications were required in the programme to assure that the assumed major-minor systems would be simulated properly. The following problems have been encountered and corrected:

- Trapezoidal channels with no bottom width cause a divide check in the calculation of the hydraulic radius if no flow is present.

- Surface channels connecting to manholes of the underground system cause a campening of surcharge and reduction of increase in nodal depth because of improper computation of available surface areas in the Model.

- The upright orifices assumed by the Model are not ideal for the simulation of catchbasins and inlet restrictions. Depressed orifices are not allowed in the present versions of the programme.

- Connections to storage facilities must be by orifices. The inlet conduits are therefore prone to instability because of their increased sensitivity to surcharging.

- Surface channels have a strong tendency to become unstable at slopes above about two percent, unless very small timesteps are used, i.e. 5 seconds or less.

With the increasing acceptance of the need to consider both major and minor drainage in urban developments, the availability of a reliable accurate runoff model that can handle such systems is important. While the use of a runoff model may not be essential for new developments where continuous road grades and overflow easements can be incorporated into the subdivision design, it is a prerequisite in the analysis of existing systems, where routing storage, inlet control and partial roofleader disconnection have to be considered. We have found that several modifications to our programme were necessary in order to obtain stable operation and to provide a realistic representation of the drainage system. It would appear desirable to improve the currently available programme such that dual drainage systems are handled properly. This is of increasing importance now that several time-sharing facilities have EXTRAN available to the general user.

BOROUGH OF SCARBOROUGH - Analysis of Impact of Urbanization on Bendale Creek Drainage Channel

The Bendale Creek is a tributary of the Highland Creek and drains a major portion of the Borough of Scarborough. The headwater of the Bendale Creek is in the Town of Markham, with about 540 acres of area north of Steeles Avenue, the north boundary of Metropolitan Toronto. The entire watershed will ultimately be urbanized, with the exception of some parkland. This puts a heavy burden on the Bendale Creek as the main drainage channel. Most of it has been improved into a formal trapezoidal channel, but some sections are still in a more or less natural state. Culverts and bridges date from quite different time periods, as do the design flows used for their sizing.

Proposals for development in the Town of Markham have initiated concern with regard to flooding downstream, following widespread flooding of streets and basements in the Borough of Scarborough in the wake of violent summer thunderstorms in 1977. Concern was raised in particular for an area known as the Subtrunk "D" Watershed, which connects to the Bendale Creek at Highway 401 and Kennedy Road. It was subsequently decided to have Gore & Storrie Limited investigate the impact on the Bendale Creek resulting from different development and storm water management assumptions in the upstream part of the watershed. Special consideration was to be given to the assessment of any adverse effects to the flood-stricken Subtrunk "D" Watershed. The study had to be on a limited scale only with lumped representation of the minor tributary watersheds and without detailed surveys of the existing channel and storm sewer systems.

After some soul-searching it was decided to use the SWMM Model with the Extended Transport Block Option for this channel analysis. This decision was based on the following requirements:

1. The channel analysis would have to consider backwater effects throughout the entire open channel system because of the numerous flow-controlling culverts and bridges.

2. The storage and routing effect of the many submerged large-size trunk storm sewers connecting to the creek would have to be considered in the analysis, requiring a dynamic model.

3. The design of the tributary storm sewer system is based on a 5-year storm recurrence; the major-storm-events to be analyzed would obviously require overland flow routing of the excess runoff to the main drainage channels.

4. The urbanized character of the watershed could best be represented by means of the Runoff Block of the SWMM Model.

5. Storm Water Management options such as Zero-Increase-in-Runoff concepts could best be modelled with the Runoff Block.

The first part of the work was to collect data on the major elements of the drainage system, such as plans of channels, bridges, culverts and major trunk sewers. A short field survey was carried out to obtain information on channel roughness, inlet and outlet transitions at culverts and bridges, as well as a very rough assessment of the overland flow routes and approximate overland channel profiles. Land-use data was obtained from aerial photographs and planning data for the future developments.

The input for the Runoff Block of the SWMM Model has been assembled in the usual manner. The 5600-acre watershed above Ellesmere Road has been divided into 35 subcatchbasins. Where storm water management techniques could be assumed, the runoff to the inlet has been controlled by means of a gutter of fixed capacity instead of a direct connection. The proportions of controlled to unrestricted flow have been taken from a storm water management proposal for the Riseborough Development in the Town of Markham, developed by James F. McLaren Limited, Toronto.

The input to the Extended Transport Block requires some special considerations. Culverts and bridges cannot be modelled as conduits unless a very small timestep is used to prevent stability problems with the relatively short lengths involved. An alternative is the use of orifices. This requires some adjustment of the orifice coefficient in order to calibrate the culvert losses to values estimated by manual calculations. A further factor to be considered is the incorrect handling of weir flow through a partially-full orifice in case of a backwater condition.

Modelling weir flow over a submerged culvert or bridge has been attempted, using the weir option of the EXTRAN Module. This has been found to be unstable unless very small timesteps are used. Large orifices with modified coefficients have given more satisfactory results. A 30-second timestep has been used successfully without instability problems after some initial adjustments.

The findings of the channel analyzing can be summarized as follows:

1. The flows in the Bendale Creek channel are controlled to a major extent by the culverts and bridges and only to a minor extent by the channel profiles, except for unimproved sections.

2. The analysis confirmed that the water levels with existing and future flows have no effect on Subtrunk "D".

3. The submerged trunk storm sewers connecting to the channel experience substantial alteration of their outlet hydrographs. This stresses the significance of the water levels in the major drainage channels for the hydraulic performance of the storm sewer systems.

4. The continuous backwater simulation provides an excellent appreciation of the dynamics of the flood hydrograph passing through the channel.

5. The evaluation of the impact of storm water management techniques on approximately 20 percent of the analyzed watershed has shown that

a reduction of the runoff to about 0.5 to 0.6 cfs per acre has an appreciable effect only for the next two miles along the main channel. The reduction of the peak flow, four miles downstream, at Highway 401, was only 10 - 15 percent.

6. When the storm pattern was assumed to move along the drainage channel at 1.5 miles per hour, the peak flow at Highway 401 increased by 10 percent.

Final Notes

In summary, the following comments can be made regarding the application of the Extended Transport Block of the SWMM Model:

1. The availability of this Model option provides the engineer in the urban drainage field with a very powerful tool to analyze large and small systems to an extent previously almost impossible because of their complexity. This applies in particular to relief and rehabilitation studies in existing systems, where the neglect of surcharge and the interaction of looped and cross-connected systems would lead to uneconomical solutions.

2. The complexity and sensitivity of the Extended Transport Module requires even more understanding and expertise than the application of runoff simulation models already do. This applies also to the more recent releases which incorporate testing features to assist the less experienced user in the determination of a timestep which should result in a stable programme execution. There still remain so many obscure reasons for the execution to stop which are difficult if not impossible to correct without some knowledge of the programme itself. The cited complexity could ultimately lead to some programme users developing conversational, interactive versions of the model, particularly in light of the comparably high cost of running Extended Transport.

3. The net cost of running EXTRAN on moderate-to-large systems with approximately 150 conduits and 100 nodes for 30 to 120 minutes of real time coverage is in the order of $100 to $300 per run. This is based on the current (1978) rates billed for use of the IBM-CMS-Time-Sharing System in Toronto, Canada. Since the programme has few major iteration loops, the cost is largely proportional to the number of elements and timesteps.

4. With the considerable cost involved it is good practice to run a drainage system, once it is set up, for a very small number of timesteps. This will provide the output of all the system data, together with remarks generated by the programme regarding revised elevations, missing links within EXTRAN and from the inlet points of the RUNOFF Block. This provides an inexpensive means to check the input data and to eliminate costly runs with crucial errors.

5. The EXTRAN Block has no continuity check which would allow a comparison of routed flow volumes with the input hydrograph from the RUNOFF Block. It is recommended, however, that the user computes a rough sum of the outfall volumes, taking into account that the Extended Transport simulation is often not covering the entire length of the input hydrograph. This comparison is particularly important if some "exotic" system is to be modelled where it may be difficult to predict the Model's handling of the data. Instabilities during the simulation are not always easy to detect in the output data, and where they occur occasionally, they may not be significantly for the final results. This can be verified easily with a continuity check.

CSO FACILITIES PLANNING USING A MACROSCOPIC MODEL (A CASE STUDY)

by John A. Lager, Vice President, E. John Finnemore, Project Manager, and George B. Otte, Project Engineer
Metcalf & Eddy, Inc., Palo Alto, CA

Simplified stormwater models can be an effective aid to facilities planning for combined sewer overflow (CSO) remedial projects. They are useful in characterizing the sources and temporal and spatial variations in wet-weather wastewater flows, identifying responses to alternative routings and control practices, assisting in the cost-effective sizing of major system components, and providing useful information for preliminary design.

In this presentation, an in-progress case study of a macroscopic model's adaptation and application in San Francisco's Wastewater Management Program is described.

MODEL ORIGINS AND INFORMATION SOURCES

Simplified stormwater models have been under development by several users and investigators since the early 1970's. Metcalf & Eddy developed its first simplified approach in 1972 in a reconnaissance level study of CSO and stormwater discharges for the District of Columbia as a necessary companion model to EPA's SWMM. This approach methodology was extended and documented for public dissemination in connection with a CSO study for Rochester, N. Y. in 1976 [1]. Land use related quality parameters (fixed) were introduced to the model in modifications for the Association of Bay Area Governments' 208 planning study in 1977. This model, MAC (for macroscopic planning model), is also scheduled for documentation and public dissemination [2].

Other discussions of the writers' viewpoints with respect to the utility of simplified models are available in EPA sponsored short course documents [3], seminar proceedings [4], and state-of-the-art assessments [5].

WHAT THE MODEL DOES

The objective of the model is to create a continuous wet-weather flow simulation, which is characteristic of the area(s) and system(s) under evaluation. The present San Francisco adaptation is identified as SFMAC.

Given a rainfall history, SFMAC computes the quantity and quality of runoff from differing land use areas, and combines these with dry-weather flow and other known lateral inflows into storage. The lateral inflows may be delayed in time, to represent flow routing. SFMAC next computes pumped withdrawals from storage, and the resulting storage volume and overflows, if any. It last computes pollutant removals by treatment of the pumped withdrawals, and the resulting quality of the discharged effluent. All computations are repeated on an hourly basis.

The program prints out the hourly status of all variables, and summaries for each day, month, water year, and subarea run. In addition, the hourly withdrawals and/or overflows may be written on disk files, for future use as input (lateral inflows read from disk files) to downstream segments.

The fundamental capabilities of SFMAC described above are depicted in Figure 1. The entire procedure can be repeated for many different subareas, all within the same run. More importantly, the basic model may be applied successively from upstream areas to downstream points, enabling many subareas and transport/storage/treatment facilities to be linked together in a manner representative of a citywide system.

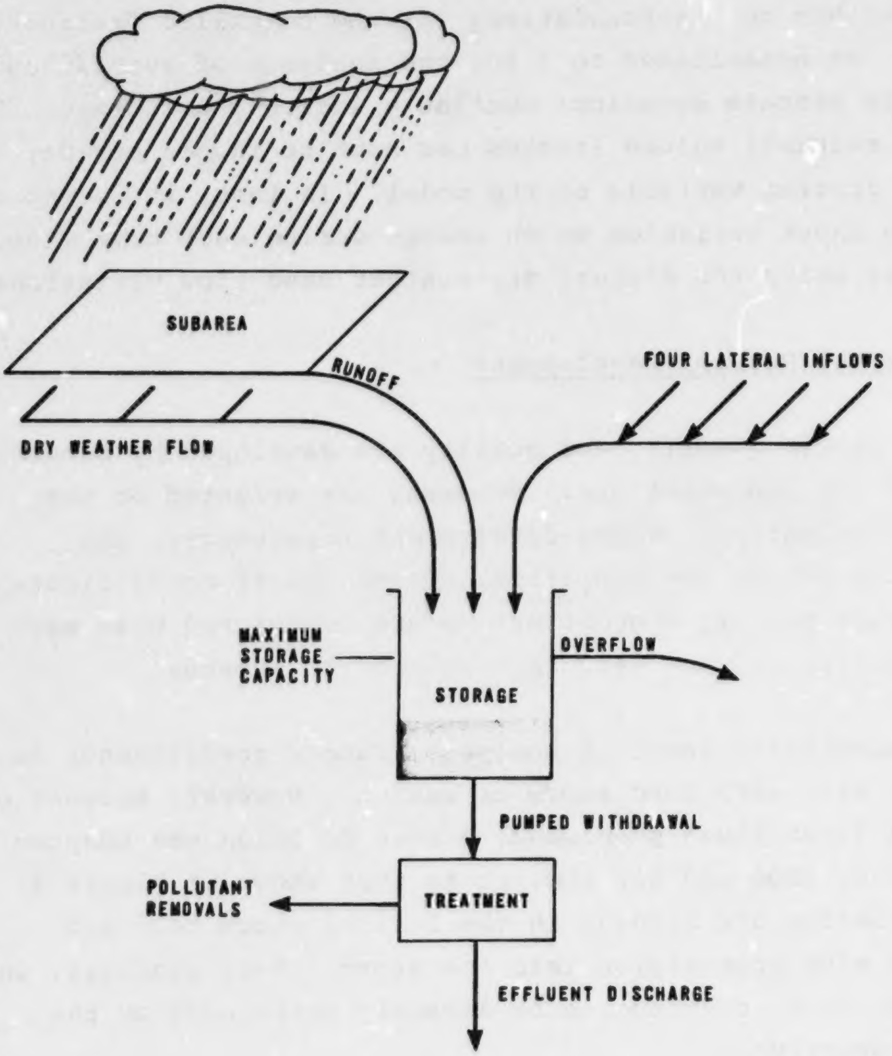

SUBAREA
RUNOFF
DRY WEATHER FLOW
FOUR LATERAL INFLOWS
MAXIMUM STORAGE CAPACITY
OVERFLOW
STORAGE
PUMPED WITHDRAWAL
POLLUTANT REMOVALS
TREATMENT
EFFLUENT DISCHARGE

FIGURE 1. SCHEMATIC REPRESENTATION OF THE SFMAC MODEL CAPABILITIES

Concept

The model has as its foundations (a) the so-called "rational method", an established tool for the analysis of runoff, and (b) the basic storage equation: outflow = inflow ± Δ storage. Time related rainfall volume (inches per hour or inches per day) is the primary driving variable of the model. In fact, it is one of only two input variables which change during each time step, the other being the diurnal dry-weather base flow variations.

Quantity and Quality Development

Surface runoff quantity and quality are developed by subarea and land use distributions. Subareas are selected on the basis of topography, slope-development homogeneity, and collection system configuration. Gross runoff coefficients and average quality concentrations are associated with each land use type and may vary from subarea to subarea.

At the simplified level of analysis, runoff coefficients do not vary with time into storm or season. However, because of observed first flush phenomena, a step function was adapted for quality (BOD and SS) similar to that shown in Figure 2. Concentrations are highest in the initial storm hour and decrease with progression into the storm. Mass loadings, on the other hand, continue to be directly influenced by the storm hydrograph.

Dry-weather flow diurnal quantity and quality variations are taken directly from the appropriate treatment plant records.

Routing and Aggregation

Flow routing is accounted for by user selected time offsets, which are fixed for a single system test configuration.

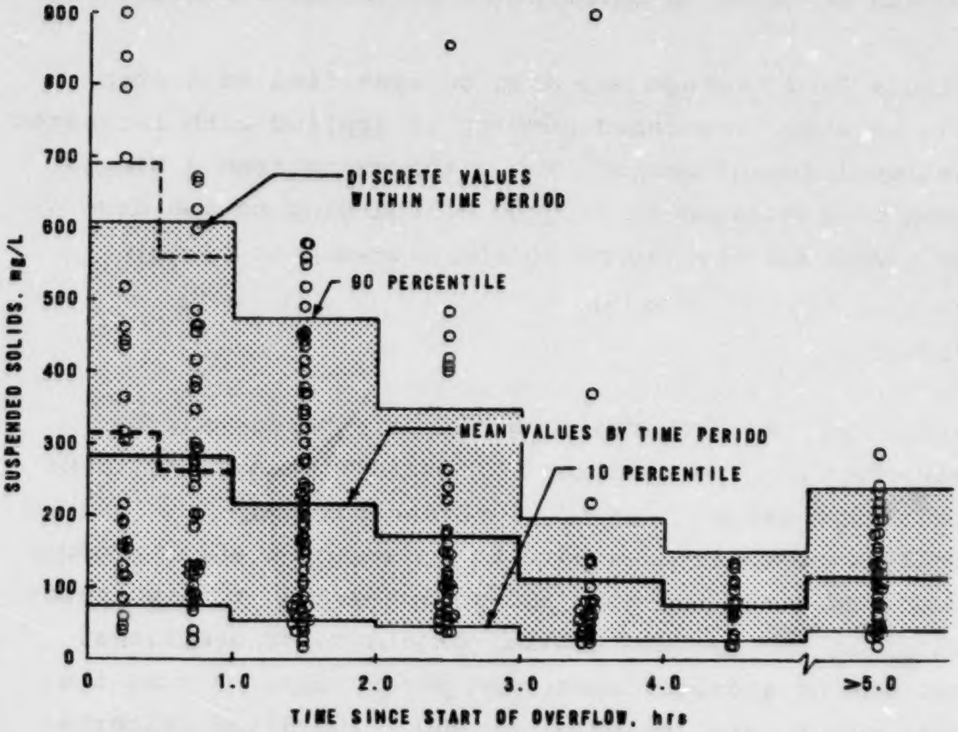


FIGURE 2. CSO QUALITY VS. TIME, SS

Aggregation of runoff from several subareas and appropriate dry-weather flows is also fixed by the text configuration.

Storage basins are treated as completely mixed each time step and a treatment quality change associated with basin detention may be specified. Treatment facilities with negligible storage are simply represented by specified quality changes, which could be based on pilot plant performance evaluations.

Withdrawals from storage may also be specified as a step function in which increased pumping is applied with increases in flow level in the basin. Two withdrawals from a single basin may be simulated (e.g., one discharging to the dry-weather plant and the second to the wet-weather plant).

Displays

The output for each subarea or group of subareas may be entirely overflow (no treatment), entirely treated effluent (all runoff processed), or both. Graphical and numerical display routines have been developed which produce continuous input hydrographs and pollutographs to any specified plant in the system, similar displays for overflows, computations of storage times, and percentages of time the treatment plants are loaded at or above specified criteria.

REPRESENTATIVE APPLICATION PROCEDURES

As an illustration of model usage in CSO facilities planning, excerpts from our in-progress San Francisco work are presented. The intent is to identify, in case study context, selected application procedures which may assist other municipalities in developing their own approach concepts.

Subdivision of Study Area

In San Francisco, the subdivision of the 25,000 acre study area, Figure 3, proceeded to two levels: major watersheds (7) and subareas (23), all of which had definable topographic/system boundaries. Major watersheds were determined by outfall consolidation projects which were in planning, design, and construction phases and, to a large extent, constituted givens in the project work. A principal project objective was to identify and detail cost-effectiveness evaluations of alternative sizing, routing, storage-treatment, and discharge schemes to solve existing wet-weather, CSO, problems. Representative schemes under evaluation are shown in Figure 4.

Rainfall

Fortunately, San Francisco has had one of the most comprehensive rainfall reporting and recording networks in the world since 1971, and a base gage which extends the period of record back over 70 years. Thirty gages in the present system telemeter rainfall instantaneously to a computer center for observation, recording, and data analysis.

To take advantage of this information, 4-year 30-gage proportioned records were developed for each of the 23 subareas and similar weighted 70-year records were made available from the single long term gage. For reasons of economy and understanding, a 4-month subset (rainfall 170 percent of normal) was selected for preliminary alternative analyses using hourly time steps.

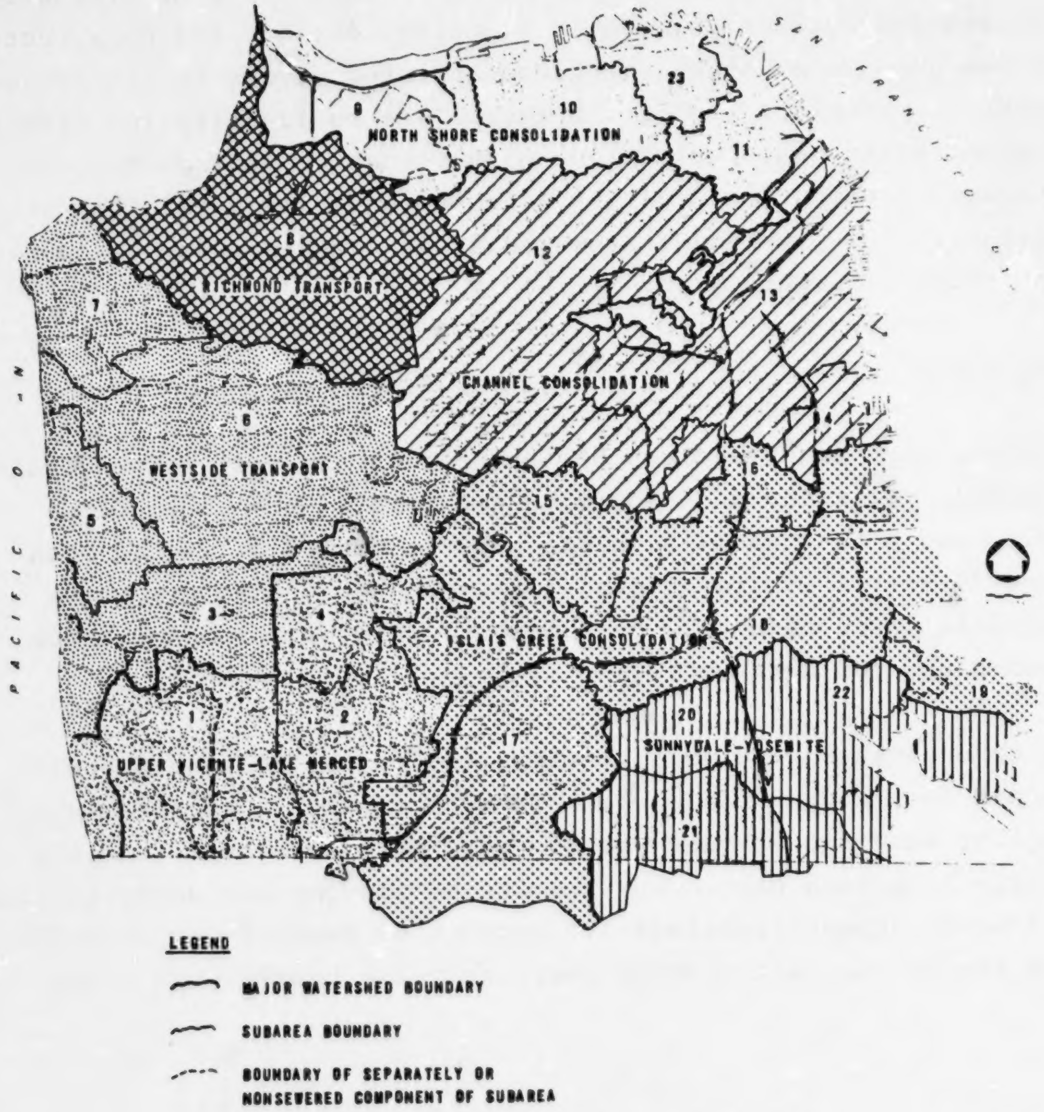
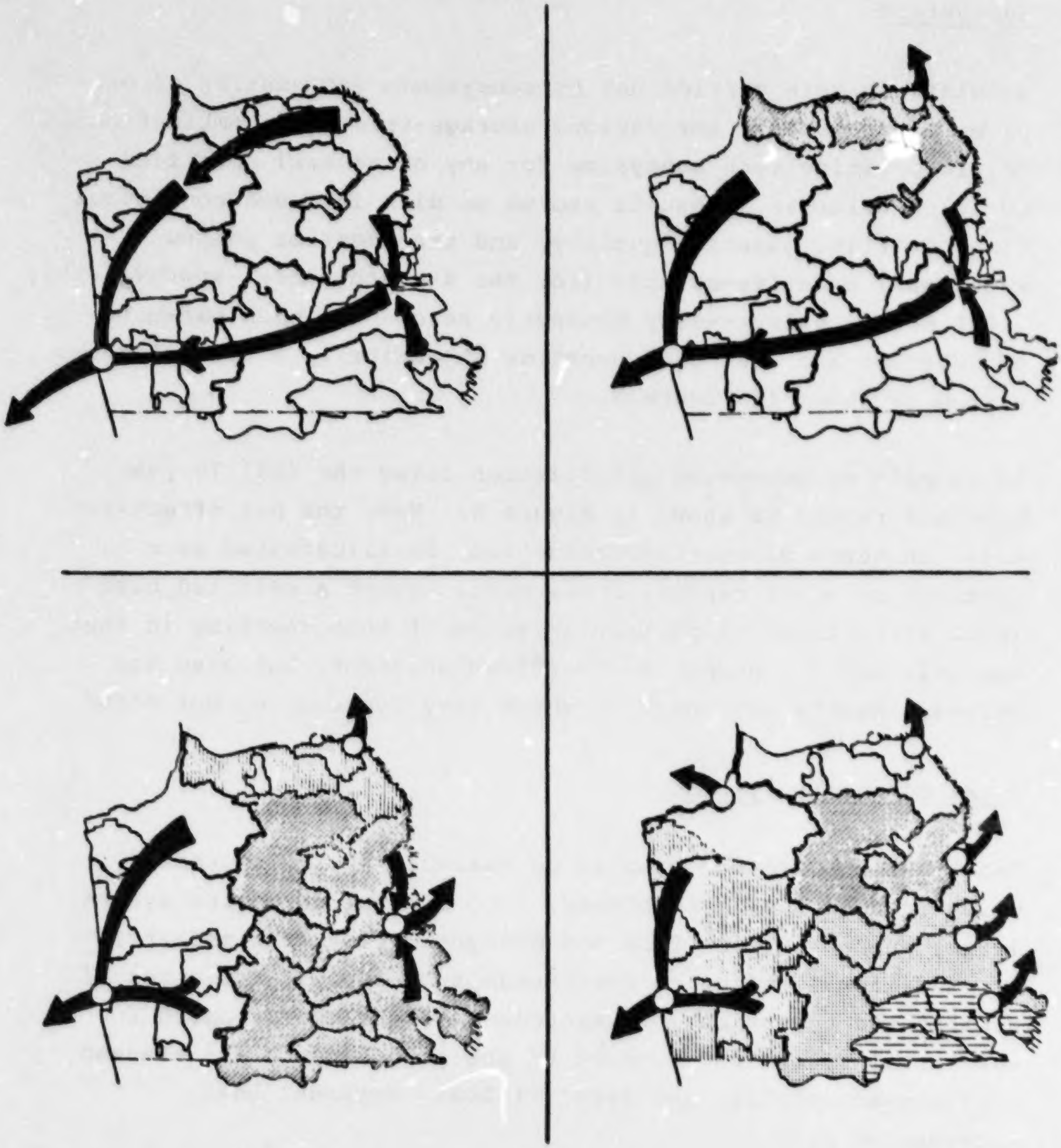

SAN FRANCISCO BAY
PACIFIC OCEAN
NORTH SHORE CONSOLIDATION
9
10
23
11
8
RICHMOND TRANSPORT
12
13
CHANNEL CONSOLIDATION
7
6
WESTSIDE TRANSPORT
5
14
15
16
3
4
ISLAIS CREEK CONSOLIDATION
18
19
1
2
UPPER VICENTE-LAKE MERCED
17
20
22
SUNNYDALE-YOSEMITE
21
LEGEND
MAJOR WATERSHED BOUNDARY
SUBAREA BOUNDARY
BOUNDARY OF SEPARATELY OR NONSEWERED COMPONENT OF SUBAREA

FIGURE 3. SUBAREAS AND MAJOR WATERSHEDS



Note: Circles represent treatment plants; arrows represent routings and discharge locations; and shading represents area tributary to a specific plant.

FIGURE 4. REPRESENTATIVE TOTAL SYSTEM WET-WEATHER SCHEMES

Subsystems

Simulations were carried out by subsystems (consisting of one or more watersheds) for various storage-treatment combinations to pre-optimize each subsystem for any of several specified overflow criteria. Results stored on disk included continuous time, overflow quantity-quality, and treatment or pumped withdrawal quantity-quality (for the 4-month subset approximately 3,000 hourly values--many obviously zeros--for each parameter of interest and for each overflow objective). A typical subsystem is shown in Figure 5.

An example of subsystem optimization using the full 70-year rainfall record is shown in Figure 6. Here the net effectiveness, in terms of overflow reduction, is illustrated as a function of equal capital investments beyond a selected base level of control. A particular value of this charting is that not only are the number of overflows apparent, but also the volumes, months and years in which they occur or do not occur.

Total Composite Systems

The final application step is to assemble the subsystems, which are now 3 or 4 in number, into a total composite system (*e*.g., pumped outflow from one subsystem plus time offset is new additional inflow to downstream subsystem). Estimates of capital and operation and maintenance costs are prepared for each total system and pursued to the environmentally-balanced, publicly-acceptable, and least(?) cost, apparent best alternative project.

Of course, optimal solutions on the total system may require use of the second or third best choice for a particular subsystem; thus several iterations are to be expected.

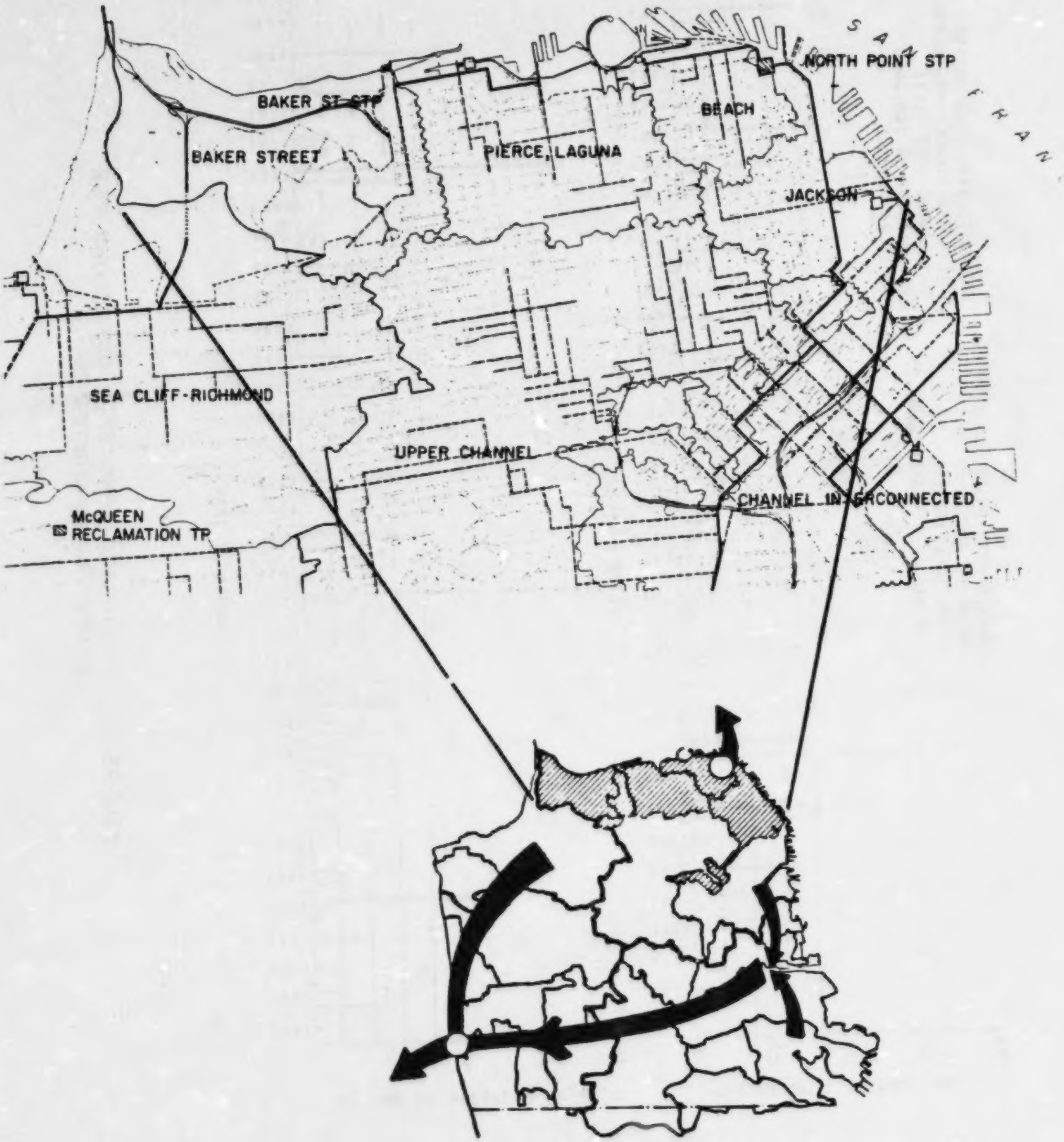

SAN FRAN
NORTH POINT STP
BAKER ST STP
BEACH
BAKER STREET
PIERCE, LAGUNA
JACKSON
SEA CLIFF-RICHMOND
UPPER CHANNEL
CHANNEL INTERCONNECTED
McQUEEN
RECLAMATION TP

FIGURE 5. NORTH SHORE SUBSYSTEM

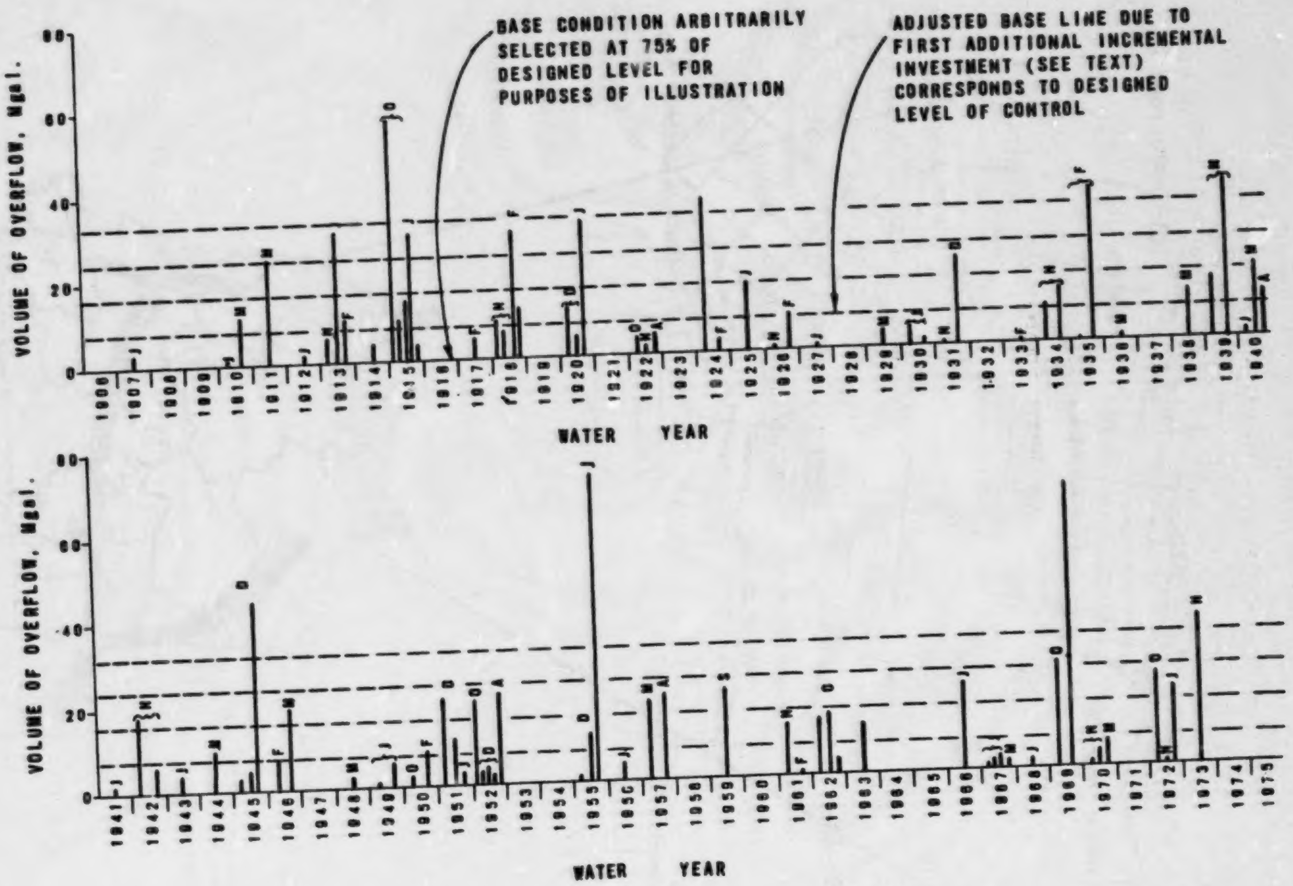


FIGURE 6. PROJECTED LONG TERM OVERFLOWS FOR NORTH SHORE SUBSYSTEM

CALIBRATION-VERIFICATION

When dealing with simplified models, what can be accomplished in the area of calibration-verification is obviously limited. Simplified models do not claim to produce precise results, as precision is intentionally sacrificed for breadth of coverage, quick setup, and low cost turnaround. Also, simplified models are generally resorted to when available data is so sparse that calibration-verification is more imaginary than real.

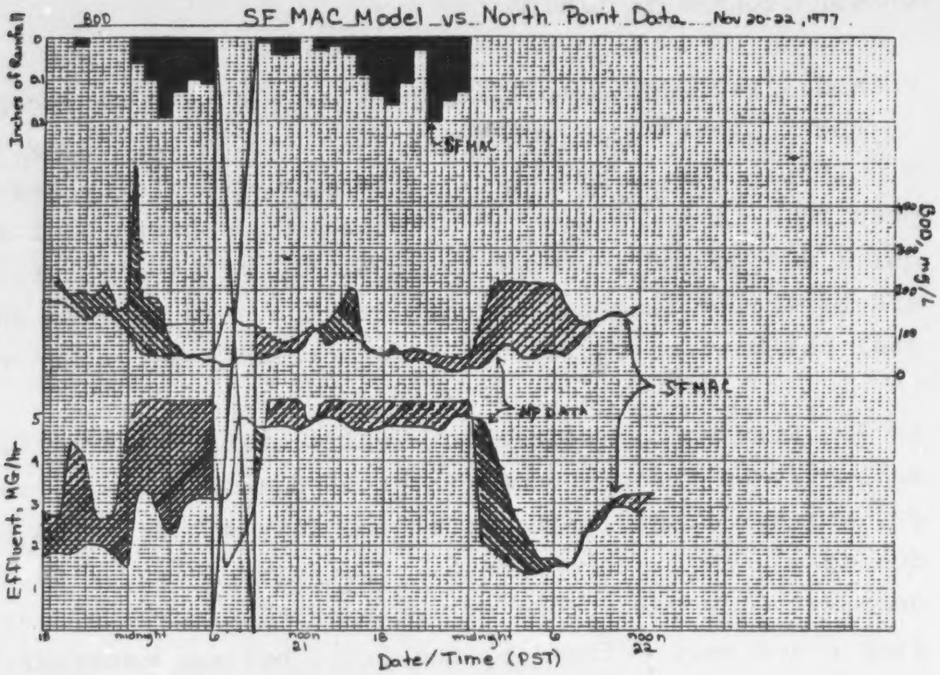
In San Francisco, calibration-verification of SFMAC was accomplished through comparisons of discrete hourly samples of dry-weather plant influents at the City's 3 existing plants during six 1977 storm events. Operating rules as to how much combined flow was pumped or transported from each area to each plant were defined by the City, but not necessarily followed by City operations staff, as their response was more dictated by the individual storm and equipment available.

Representative results for one storm at two plants are shown in Figure 7. Correlation coefficients after calibration for the 3 plant, 6 storm, several hundred element matrix ranged from 0.6 to 0.9 for flow, 0.4 to 0.7 for BOD concentration, and 0.3 to 0.5 for SS concentration.

UTILIZATION IN PLAN DEVELOPMENT

SFMAC's adaptation and application in San Francisco's facilities planning process already has yielded many tangible results, including the following:

- It has provided a common denominator for evaluating widely differing alternatives;



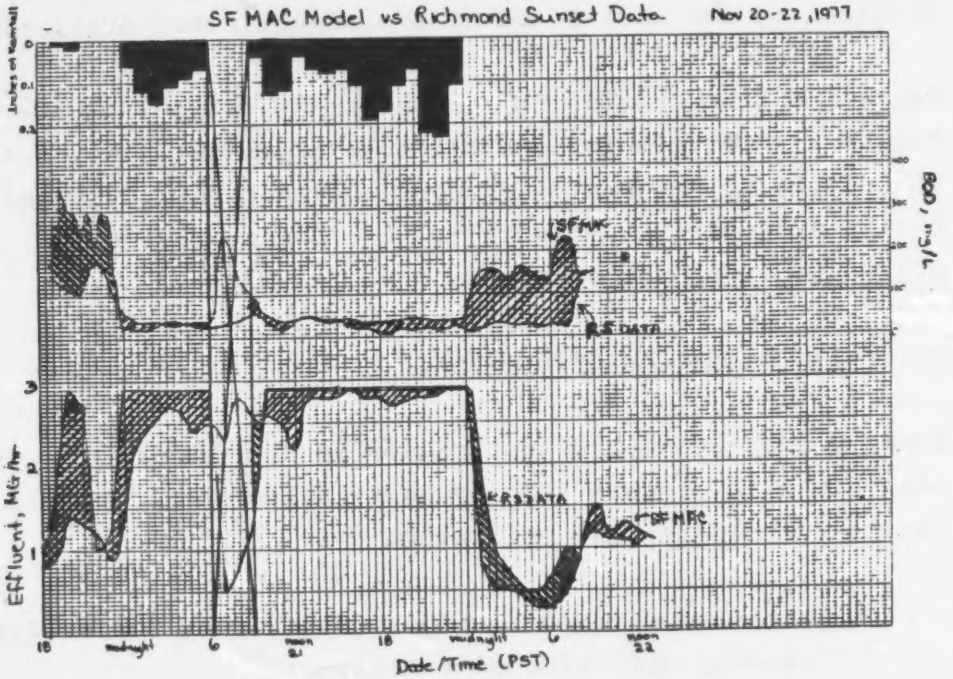


FIGURE 7. MODEL CALIBRATION EXAMPLE RESULTS

- It has provided information for preliminary sizing for all major system components for feasible systems and target levels of control;
- It has identified significant locations, times, and volumes of both treated and untreated discharges;
- It has provided detailed potential plant influent characterization, Figure 8, for synthesizing wet-weather flows for pilot plant operations;
- It has provided valuable information on travel and system dewatering times to indicate potential odor problems;
- It has provided a tool for estimating annual energy and chemical needs and estimates of sludge production;
- It has served as an important tool in the narrowing of systemwide alternatives from 40 to 10, and will continue to be of assistance as alternatives are further reduced until the final selection of the apparent best alternative; and finally
- It is providing an effective tool for conveying the complexities of such system evaluations to the public, the engineering community, and the decision-makers.

In conclusion, simplified stormwater models, professionally applied, provide an understanding of wet-weather system operations and requirements through tools which are highly flexible, economically applied, and valuable to decision-makers.

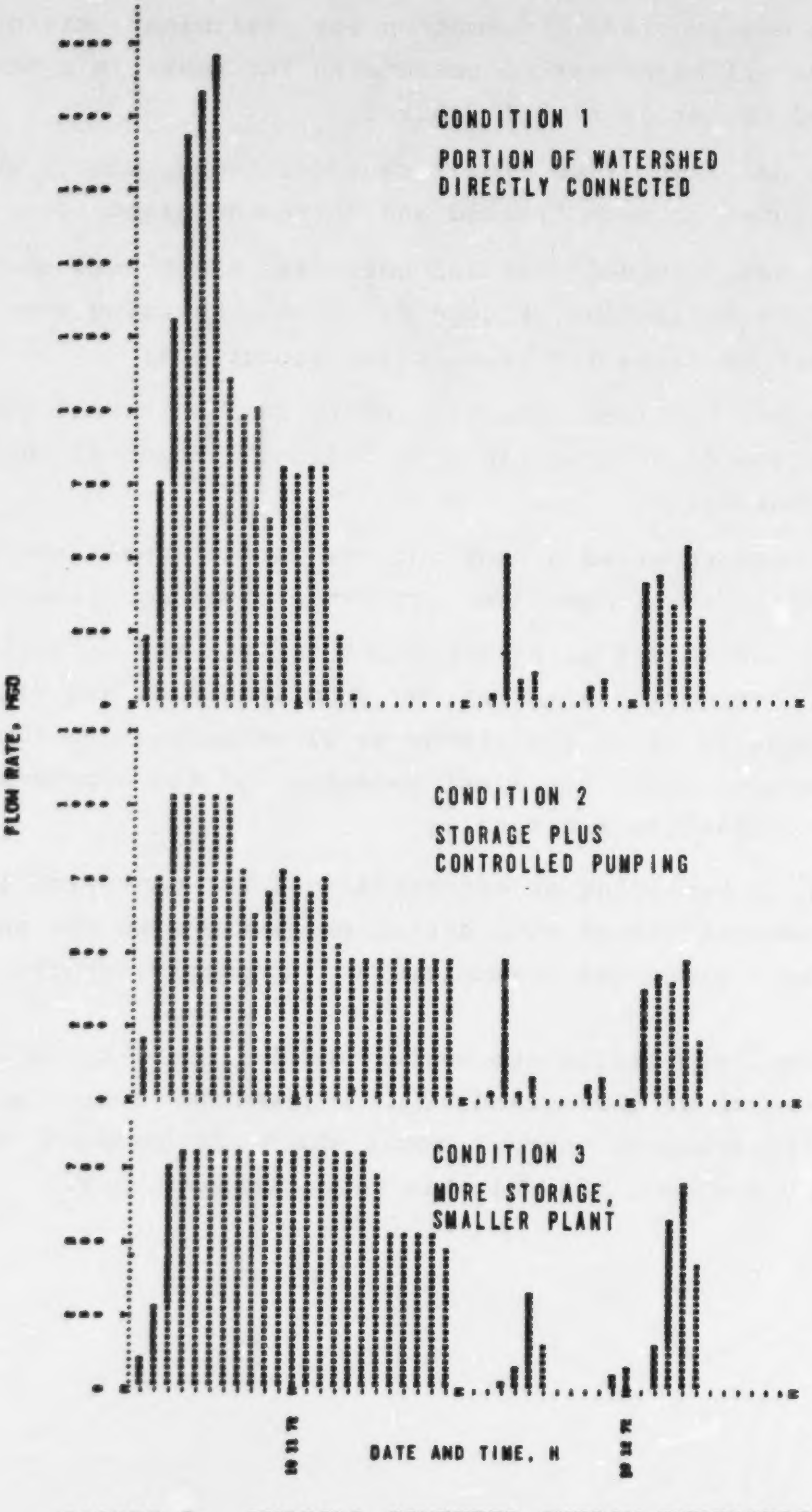


FIGURE 8. TYPICAL COMPUTED INFLOW HYDROGRAPHS TO WET-WEATHER PLANT

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STORMWATER MANAGEMENT MODELLING AND LAND DEVELOPMENT PROJECTS

PAUL E. WISNER
JAMES F. MACLAREN LIMITED

INTRODUCTION

Several Stormwater Management Studies have recently been conducted by James F. MacLaren Limited in areas having potential restrictions to development. Modelling conducted under those circumstances, although requiring more time than traditional analysis, helped to define drainage alternatives and trade-offs, which make development of certain areas acceptable. In addition, it contributed to better planning and easier approval processes.

Three main types of restrictions related to flood and runoff control from developments are: development in floodplains, runoff increases and pollution caused by runoff. At present, in Canada, most provinces have various criteria for floodplain control, but only some of the provinces are in the process of developing criteria for runoff quantity control. Runoff quality constraints have been considered only on rare occasions when dealing with environmentally very sensitive areas. Lately, at the municipal level, there has been a significant increase in interest in all three above aspects; a number of progressive municipalities have started to require some form of runoff control despite the lack of provincial regulations. More recently, a number of other agencies have also become involved in stormwater management as, for example, some conservation authorities such as the Metro Toronto and Region Conservation Authority. The following typical examples, which involve modelling related to land development, lead to some general considerations regarding the implementation of Stormwater Management (Table 1).

Zero Runoff Increase

In the absence of specific runoff control regulations, the requirement of "zero runoff increase" has evolved, mainly as a consequence of potential downstream flooding. A typical example is the new 160 acre residential development in Markham, draining to an area in Scarborough severely afflicted by basement and street flooding at present. A traditional drainage system in the new development would have entailed a contribution of $1.5 million towards increasing the capacity of the downstream storm trunk sewers, as well as delays related to the implementation of relief sewers in a large area in Scarborough. As against this, adoption of runoff control for both the minor and major systems has resulted in maintaining the 1/5 year peak flow at the pre-development level, and in reduction of overland flow to Scarborough during major storms. The control system consists of an underground over-designed pipe, which acts as a storage unit for the 1/5 year storm runoff, and the use of parks for dry detention areas, which only operate during major storms (see Appendix). The analysis accounted for time lag between the release from the various storage units and smaller areas without control, combination of overland and pipe flow, estimation of surcharges in various parts of the system, etc. This level of sophistication is, of course, only possible by using computer programs such as SWMM 6 (WRE). The project went into the detailed hydraulic design of easements leading overload flow to detention areas, the outflow facilities at various elevations from the detention areas, orifices for flow reduction, etc. The extra cost of the stormwater management system consisted mainly in the oversizing of a trunk system, estimated to cost about $350,000.

A similar project, involving a combination of underground storage and different surface detention facilities for control up to the 1/100 year flow is presently being studied for a development in Newmarket, where other drainage alternatives were not acceptable to the municipality.

Optimum Use of Existing Storm Sewer Capacity

A frequent problem with new developments is inadequate capacity in existing trunk storm sewers. As an example, for a large development in North-east Edmonton, this constraint would have required an initial investment of about $15 million for a new trunk sewer. This large cost was caused by the distance to the receiving river, and by the need to cross a ridge separating the development and the river. The existing trunk sewer, located south of the development, operates at full capacity for the 1/5 year storm. In addition, there were major concerns in connection with the effect of new development on increasing overland flow to the developed neighbouring areas and, consequently, runoff controls were considered for storms up to the 1/100 year level. Considerable storage (up to 720 acre-feet) is required to reduce the 1/100 year storm flow from 2500 cfs to 50 cfs, a runoff rate which could be directed into the existing trunk sewer during a storm. According to our present studies, this storage could be implemented by means of 19 urban lakes, interconnected by a system of relatively small pipes and swales. In one of the alternatives, buffer lakes would be located at the downstream end of the system and control gates would regulate and direct the outflows from these lakes into the existing trunk sewer only at times when capacity became available. If the cost of land associated with dedication of 90 acres for the lakes is neglected, this alternative would cost only about $4.5 million. The land, however, has to be dedicated in addition to that considered for parks. However, the lake system could be implemented over a period of 10-15 years. The resulting deferment of large initial costs is decisive for the economics of the whole project.

The performance of the drainage system with lakes over a period of time was analysed by continuous simulation after screening Edmonton precipitation records for a period of over 70 years. It is interesting to note that the critical situation is a particular sequence of two storms. Other aspects included in the hydraulic analysis were: effects of storms with various durations, comparison of summer storms and snowmelts, analysis of overland flow in the neighbouring areas, etc. The analysis involved using the computer programs STORM and HYMO, for hydrologic work, and the WRE Transport Model, for trunk sewer operation.

RUNOFF QUALITY MANAGEMENT

Water quality constraints from storm runoff appeared in conjunction with a number of very specific situations. As an example, the construction of storm outfalls to a large creek in Edmonton has been delayed because of concerns expressed by powerful citizen groups. The only other drainage alternative was connection to a distant trunk sewer. A James F. MacLaren Limited study, presently under review, has examined the effect of a storage pond designed to significantly reduce first flush effects. The study compared water quality measurements in the stream with those simulated for the development with and without storage.

Another example of public concern related to the effects of urbanization that involves a new development in Ancaster, Ontario. In this case, the frequency of overflows to downstream properties was significantly reduced by initially diverting the entire drainage system into a detention pond which, in addition, functions as a sediment trap and then releasing the outflows into a larger watershed. In both situations described above, the STORM model and the analysis of water quality measurements were used to define impacts and compare alternatives.

Water quality considerations also played a significant role in planning studies for comparing the effects of various stormwater management alternatives (storage, street sweeping, etc.). The first example is a stormwater management demonstration study in St. Thomas, Ontario (described in a separate paper). A similar study, but on a larger scale, was carried on for the City of Edmonton. The main conclusion related to stormwater management was that the total pollution load from the new developments would not have a significant effect on the water quality in the North Saskatchewan River, and that other pollution abatement problems related to this study have a higher priority.

A watershed assessment, now under review, has been carried out for a 40 sq. mile watershed tributary to Lake Simcoe, where various development and stormwater management alternatives were assessed from the viewpoint of flood control, runoff increase and water quality in a receiving stream and the lake. It was concluded that stormwater management was required mainly for erosion control and more advanced phases of development.

The impact of areawide studies of this nature on the land development industry is very important. There are, at present, municipalities in the Thames River, Credit River and other watersheds where development has been frozen by regulatory agencies because of lack of assimilative capacity in the receiving streams. Modelling studies which include urban runoff effects, alternatives for disposal of sewage, etc., may be beneficial for a more scientific land use planning and consideration of more flexible restrictions in conjunction with stormwater management.

DIFFUSED VERSUS WATERSHED STORMWATER MANAGEMENT

The example described in the Appendix refers to stormwater management on a given property. This leads to diffused stormwater management which may be implemented relatively easily but may not necessarily optimize the trade-off between environmental, aesthetic, economic and runoff control aspects. Formulation of a master plan acceptable to several land owners with conflicting interests, and assessment of levies for detention facilities requires regulatory experience; a good understanding of the scope and limitations of stormwater management by both the municipality and the developers is necessary. Attempts in this direction have been made by Markham for a watershed with an area of 3.5 square miles which will undergo nearly complete urbanization. Various alternatives for diffused and watershed management were assessed from the viewpoint of flood control, erosion control, environmental impact, maintenance, economics and acceptability to the developers. The final alternative involving two wet ponds and a number of dry detention areas seems to be amenable to the various parties involved. Modelling of flows with frequencies between 1/5 years and 1/100 years was carried out using HYMO and SWMM 6 using a coarse model; routing through the numerous storages designed for stormwater management or created by existing culverts was included. Detailed modelling, similar to the one carried out for Riseborough, will be considered as the various subdevelopments are submitted for approval.

FINAL REMARKS

The various examples described above indicate a large number of combinations of physical constraints and simulation problems involving the whole array of modelling aspects such as storm selection, selection of models for planning and design purposes, surcharge effects, combined overland and pipe flows, calibration of water quality models, etc. In many situations the use of models has helped the developers towards a better understanding of the real problems, solving some of the approval processes and achieving more economical alternatives. While there is still some confusion regarding stormwater management alternatives, for example, wet ponds against dry ponds, or underground detention and cost sharing, it seems that there are no major problems in understanding the basic concept and in co-operating with modelling as a requirement for drainage studies. It seems, therefore, that there is nothing to prevent the use of a more rigorous analysis in the planning and design of new developments and that the experience of progressive municipalities should be studied.

Figure 1 shows the traditional Rational Method approach. As indicated in Figure 1, many of the runoff increase criteria and simplified methods of computation lead to similar straightforward design procedures. Elements for site tailored terms of reference which would give the developer more flexibility and municipalities better overall alternatives are outlined in Table 2. Drainage criteria, in this case, are the result of preliminary planning rather than the input.

TABLE 1

SPECIFIC REQUIREMENTS FOR RECENT SWM STUDIES

RISEBOROUGH - MARKHAM (160 ACRES)	ZERO RUNOFF INCREASE FOR ALL STORMS 1/5 TO 1/100 YEAR & HISTORICAL
NEWMARKET 35-2 (85 ACRES)	SAME AS ABOVE + FLOOD DAMAGE REDUCTION IN SMALL EASTERN CREEK + CONTROL FOR OTHER PROPERTIES
NORTH-EAST EDMONTON (2000 ACRES)	DRASTIC FLOW REDUCTION 2500 TO 50 CFS IN EXISTING STORM SEWER
(300 ACRES) SOUTH EAST EDMONTON	CONTROL OF POLLUTANT LOADING IN RECEIVING CREEK AND EROSION CONTROL
SOMERSET - ANCASTER	PROTECTION OF TROUT PONDS IN DOWNSTREAM PROPERTIES
UNIONVILLE - MARKHAM	PROTECTION OF WATER QUALITY IN TOOGOOD LAKE AND EROSION IN BRUCE CREEK
BARRIE ANNEXATION	CONTROL OF POLLUTANT LOADING TO LAKE SIMCOE

TABLE 2

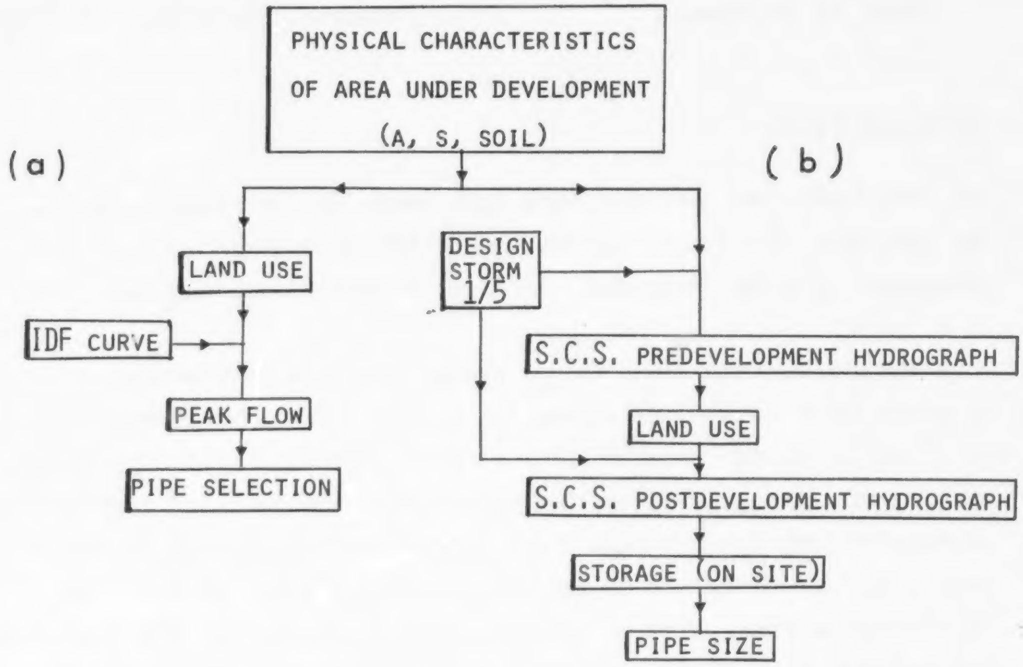
PRELIMINARY PLANNING

(NO PRESELECTED CRITERIA)

ALTERNATIVES	. DIFFUSE VS. WATERSHED . 1/5 TO 1/100 YEAR . ZERO INCREASE, LESS OR MORE . MAJOR SYSTEM?
CRITERIA	. RUNOFF INCREASE . FLOODING . POLLUTION ASPECTS . OTHER SENSITIVE ENVIRONMENTAL FACTORS (STREAM VALLEY)
CONSTRAINTS AND PRACTICABILITY	. ECONOMICS . EXISTING STRUCTURES . JURISDICTIONS . STATUS OF PLANNING . MAINTENANCE
CONFLICTS	. LAND DEDICATION . VARIOUS OWNERS . LEVIES

RESULT: CRITERIA AND
TERMS OF REFERENCE

FIGURE 1


PHYSICAL CHARACTERISTICS
OF AREA UNDER DEVELOPMENT
(A, S, SOIL)
(a)
(b)
LAND USE
DESIGN
STORM
1/5
IDF CURVE
S.C.S. PREDEVELOPMENT HYDROGRAPH
PEAK FLOW
LAND USE
PIPE SELECTION
S.C.S. POSTDEVELOPMENT HYDROGRAPH
STORAGE (ON SITE)
PIPE SIZE

SOME REGULATORY TRENDS:

- SIMPLIFICATION OF CRITERIA
- SIMPLIFICATION OF COMPUTATIONS
- SIMPLIFICATION OF REVIEW PROCESS

APPENDIX

SELECTION AND IMPLEMENTATION
OF DRAINAGE DESIGN MODELS IN MARKHAM

D. Mukherjee
D. Keliar
Town of Markham

P. Wisner
S. Vatagoda
James F. MacLaren Limited

INTRODUCTION

In the last few years there has been an increased trend to replace the Rational Method with more sophisticated drainage design methods based on computer modelling.

The Canadian Urban Drainage Committee has sponsored a significant number of model-oriented research projects and municipalities and land development consultants have realized that modelling helps reduce the cost of providing drainage facilities not only by its more precise flows, but also by allowing a better understanding of how the drainage system works. Increased pressure for the control of runoff flow increases and quality has induced the design of various storage facilities, many of which could not be sized by the Rational Formula.

While some large municipalities have in-house modelling expertise, most of them have a user-oriented understanding of modelling and prefer the use of external consultants. A new trend is also that of joint ventures in which the consultant having the responsibility for the overall drainage plan subcontracts modelling to a specialized group.

As a medium-sized municipality, Markham engineers, after a period of training by participation at various seminars and workshops and study of experience in other cities, have decided to use specialized modelling consulting services. It was recognized that despite the significant improvements in the applications of simulation, we are still in a learning phase in which some of the pitfalls are not avoided and some of the methods are misused.

While the stormwater management concepts have become quite popular, there are aspects in both quantity and quality simulation where extreme care should be exercised in the use of models.

Criteria for Model Selection

The channel routing routines of some popular models such as ILLUDAS or HYMO are very simplistic and cannot simulate, for example, such conditions as surcharge effects, backwater in pipes, etc. However, many drainage alternatives used in stormwater management require these capabilities for realistic modelling. For example, relief sewer studies in Winnipeg and Edmonton have indicated that the cut-off relief sewer may lead to significant savings. This alternative can be simulated only by models having a sophisticated TRANSPORT routine such as that in the WRE model developed by Water Resource Engineers, Inc. and presently available also in SWMM 6.

Dry or wet ponds may lead to temporary surcharge and backwater effects in the pipe system which are severely distorted by some of the "simpler" models (which may be appropriate for other studies).

The problem becomes even more complex if underground or pipe storage is considered. Surcharge of trunk sewers may lead to increased surcharge of laterals and flooding of foundation tiles. The levels and limits of surcharge have, therefore, to be determined only by a detailed analysis using a model which simulates the above aspects realistically.

Another problem in storage analysis which seems to be ignored in many simplistic simulations, is the effect of routing on the coincidence or lagging of peak flows from various tributaries. Modelling in a situation with several storage units as shown in Figures 1 and 2 should, therefore, evidence these effects by appropriate routing procedures. If the analysis is conducted for a major system, channel routing should consider both street and pipe flow and this again is possible only by the more complex TRANSPORT routine.

If runoff control is required in connection with flow limitations to pre-development conditions, one should appreciate the fact that flows from small rural watersheds may be more approximate than those from urbanized areas with a high percentage of imperviousness. Comparison between two models may be useful.

Proper selection of input data may be as important as model selection. Many studies are still conducted with "design storms" on the incorrect assumption that the storm frequency is the same as the flow frequency, while the latter is strongly influenced by antecedent conditions. In many storage applications, the critical event may not only be a storm with a critical duration, but may also be a sequence of storms. Consequently, while design storms are useful in preliminary studies, real storms have to be considered in any detailed analysis.

A Case Study

Some of the general principles previously discussed will be illustrated in the following brief description of the runoff control analysis carried on for a new development in Markham. The modelling analysis considered both the pipe or convenience system and the overland flow or major system. Flow directions in the major system are indicated in Figure 1.

- Pre-development flows for the 160-acre area were checked by comparing results with HYMO and SWMM and the results are given in the following table:

Flows for Pre-Development Conditions (in cfs)

Design Storm Frequency	SWMM Simulation	SCS (HYMO) Simulation
1/5 year	50	51
1/10 year	67	79
1/25 year	93	118

- Post-development flows for the 1/5 year frequency for two-thirds of the area are controlled by means of underground storage.
- Flows for 1/25 and 1/100 year storms are controlled by means of two dry detention ponds A and B located in parks (Figure 2). Street grading, and location of easements were carried out jointly by the modellers and the consultant working for the developer, Fred Schaeffer and Associates (in close cooperation with the town engineers). By close cooperation with the Planning and Parks Department, it was possible to locate the surface storage in parkette and recreational areas. Dry ponds are operated only for storms with a frequency of less than 1/5 years and only for a few hours. Under these circumstances, maintenance problems are minimal. Grading and landscaping of dry ponds was designed by Daniel Farb, Architect, in close cooperation with the Parks Department.

A detailed analysis carried out by means of the WRE model has indicated that surcharge is limited to a small section of the trunk sewer and will be less than 4 feet for the 1/5 year storm. Figure 3 illustrates the need to define the lag in time between flows from the two parts of the watershed.

Pre- and post-development flows are compared in the following table. Simulations were carried out not only for design storms but also for real storms with a frequency of 1/25 years.

Storm Recurrence Interval	Pre-Dev. (cfs)	Post-Dev. (no Stormwater Management) (cfs)	Post-Develop. with Stormwater Management (cfs)
2 yr.	32	110	60
5 yr.	50	168	75
10 yr.	67	220	85
25 yr.	93	275	95

Since the downstream watershed is under the jurisdiction of the Borough of Scarborough, the design and computations were submitted to this municipality for review at preliminary and final stages. Special modelling aspects were reviewed independently by Scarborough with the assistance of a specialized consultant.

It is obvious that by using modellers and landscapers in a drainage project, the cost of design will be increased as compared to a traditional method. It was found, however, that this additional expense is small compared to the savings in downstream channelization resulting from flow reductions. Additional intangible benefits are increased safety against flooding caused by overland flow and the well-known environmental advantage of runoff control.

On the other hand, the detailed modelling effort and the checking of results by various models and independent consultants gave both municipalities the confidence that future operation of the system was properly predicted.

It may be interesting to note that even if initially there were some concerns related to the difficulties caused by a change in computation techniques, cooperation between town engineers, modellers and consultants working for developers proved that modelling could be easily implemented and assimilated, even for a project with a higher than usual degree of complexity.

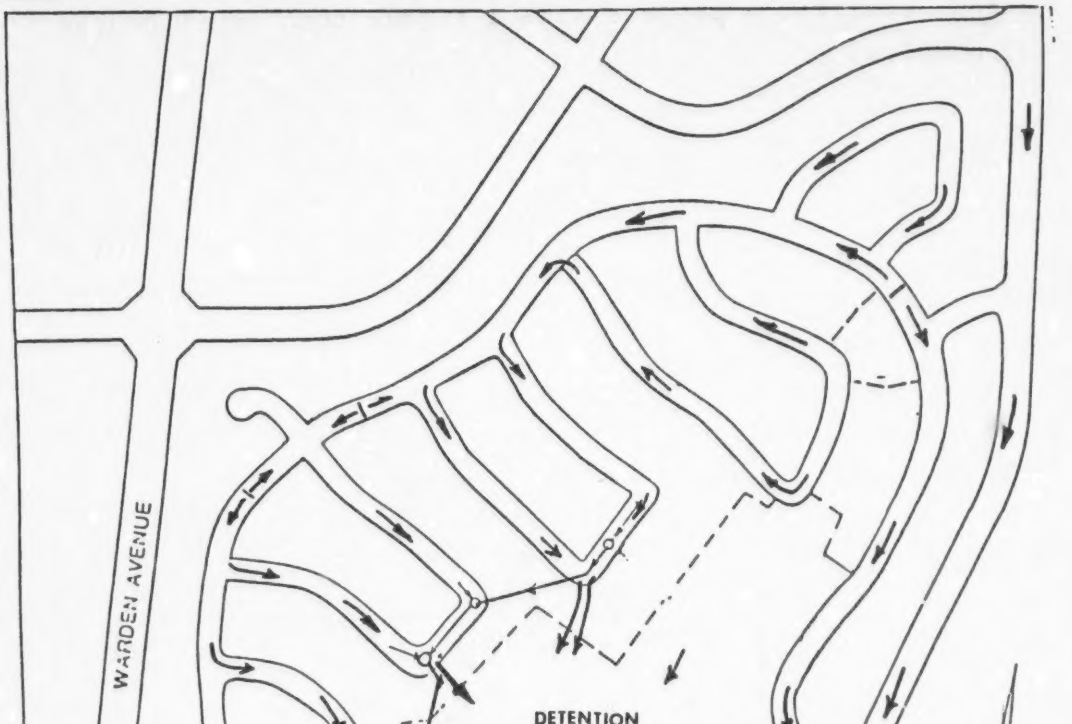
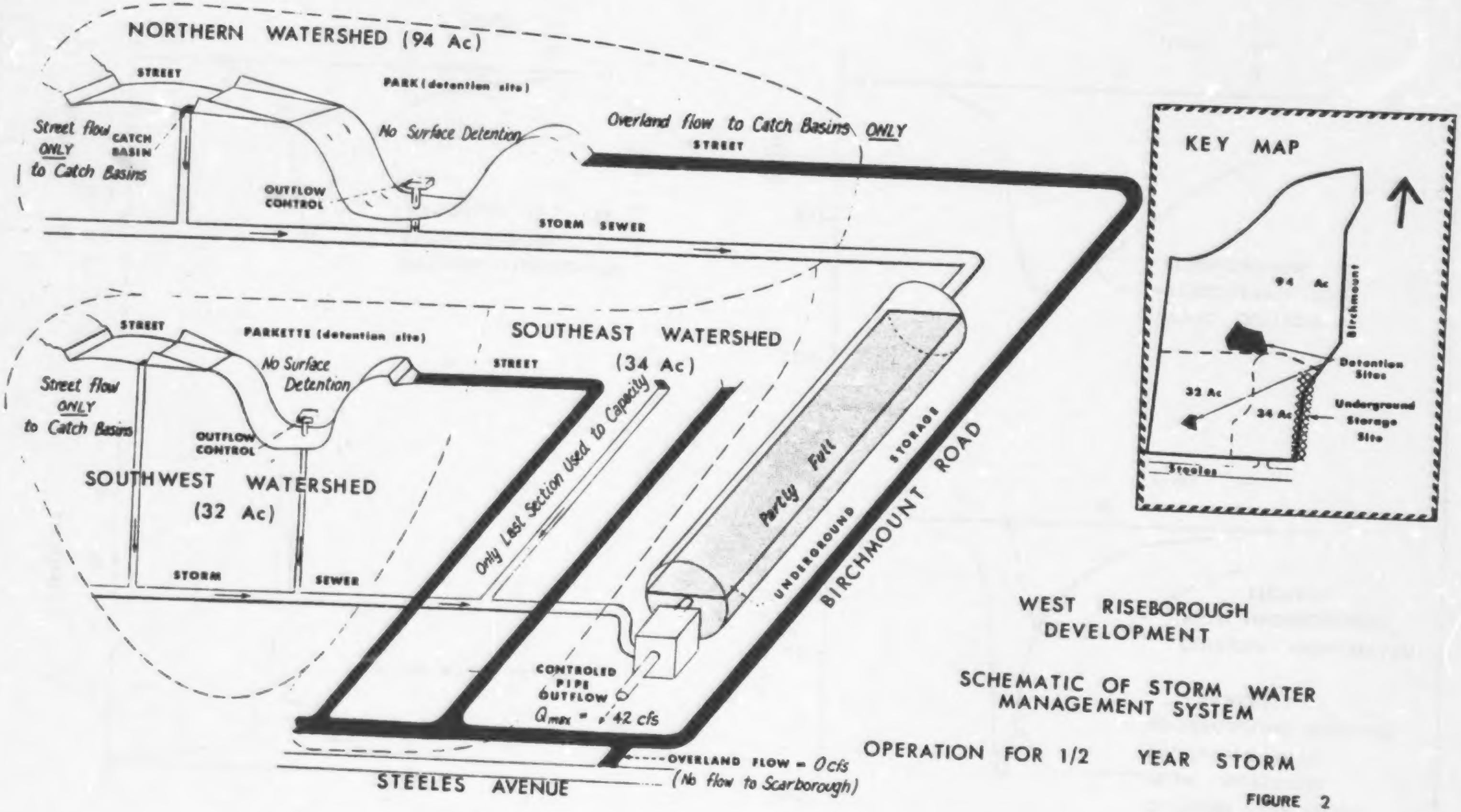

WARDEN AVENUE
DETENTION SITE
DETENTION SITE
N
WEST RISEBOROUGH RESIDENTIAL SUBDIVISION
MAJOR SYSTEM
UNDERGROUND STORAGE
0 200 400 FT
0 100 M
STEELES AVENUE

FIG: 1


NORTHERN WATERSHED (94 Ac)
STREET
PARK (detention site)
Street flow ONLY to Catch Basins
CATCH BASIN
No Surface Detention
OUTFLOW CONTROL
Overland flow to Catch Basins ONLY
STREET
STORM SEWER
SOUTHEAST WATERSHED (34 Ac)
STREET
PARKETTE (detention site)
STREET
Street flow ONLY to Catch Basins
No Surface Detention
OUTFLOW CONTROL
SOUTHWEST WATERSHED (32 Ac)
Only Last Section Used to Capacity
Partly Full
UNDERGROUND STORAGE
BIRCHMOUNT ROAD
STORM
SEWER
CONTROLED PIPE OUTFLOW
Q max = .42 cfs
OVERLAND FLOW = 0cfs
(No flow to Scarborough)
STEELES AVENUE
KEY MAP
94 Ac
Birchmount
Detention Sites
32 Ac
34 Ac
Underground Storage Site
Steeles

WEST RISEBOROUGH
DEVELOPMENT

SCHEMATIC OF STORM WATER
MANAGEMENT SYSTEM

OPERATION FOR 1/2 YEAR STORM

FIGURE 2

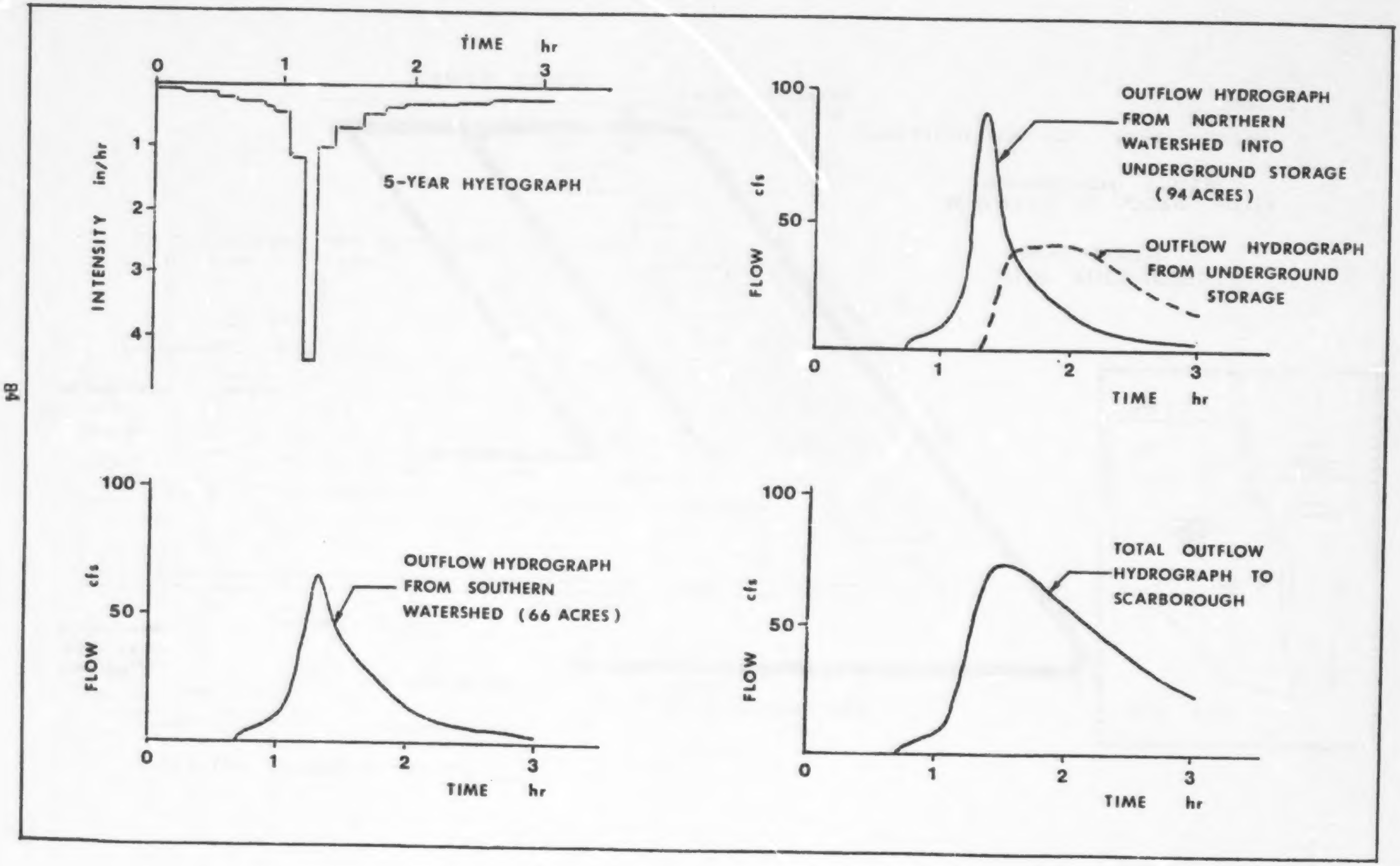


FIG. 3 FLOW HYDROGRAPHS OF WEST RISEBOROUGH DEVELOPMENT FOR 1/5 YEAR STORM

APPLICATION OF STORM TO ASSESS THE IMPACT OF AN URBANIZING AREA

Presented by

C. Brcic, P.Eng.

Gore & Storrie Limited
Ottawa, Ontario

SWMM Users Group Meeting
Ottawa, Ontario May 4-5, 1978

INTRODUCTION

The Rideau River, a major watercourse of Eastern Ontario, has a drainage area of about 1,580 square miles and passes through the central urban area of the City of Ottawa, where it drains into the Ottawa River. The Ontario Ministry of the Environment has recognized the Rideau River as sensitive with respect to its assimilative capacity and as such has requested that comprehensive reports be prepared on the effects of storm water discharge to the River from new developments.

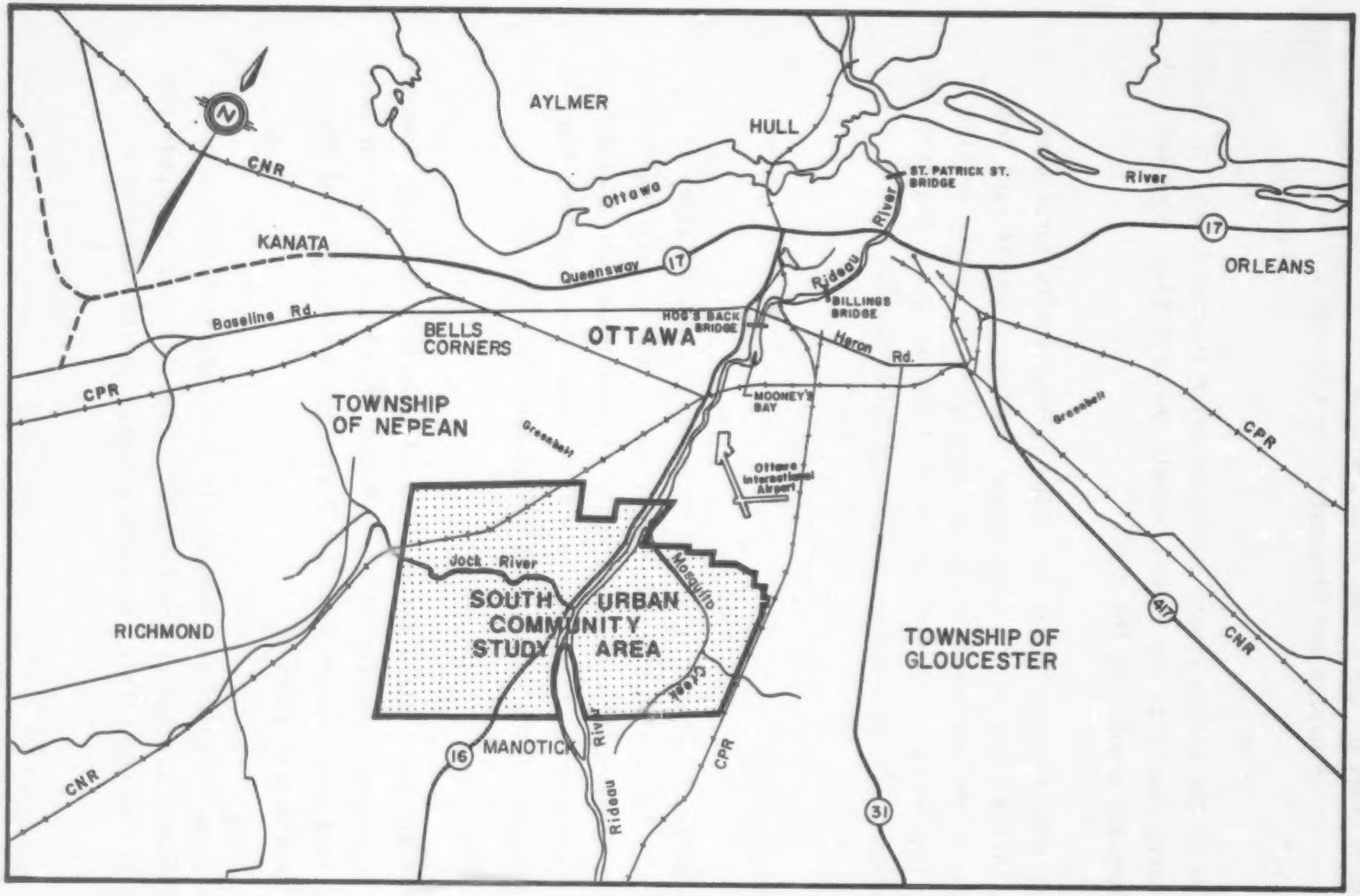
This paper deals with the report on storm water management for the proposed South Urban Community, which is the largest proposed community from which all storm water drainage would be to the Rideau River, either directly or indirectly.

PURPOSE OF STUDY

The proposed development of the South Urban Community and the policy of the Ministry of the Environment with respect to any discharges to the Rideau River, required that a detailed analysis be undertaken in order to determine the quantity and quality aspects of the runoff generated within this proposed development. The study required detailed analysis and recommendations for a storm drainage system and method of storm water management so as to meet quality objectives provided by the Ministry of the Environment.

STUDY AREA

The study area, as shown on Figure No. 1, straddles the Rideau River south of the N.C.C. Greenbelt which is a designated area restricting urban development, surrounding the Ottawa urban area along with the


N
AYLMER
HULL
Ottawa
River
ST. PATRICK ST. BRIDGE
CNR
KANATA
Queensway
17
ORLEANS
Rideau
River
BILLINGS BRIDGE
HOG'S BACK BRIDGE
Baseline Rd.
BELLS CORNERS
OTTAWA
Heron Rd.
CPR
TOWNSHIP OF NEPEAN
Greenbelt
MOONEY'S BAY
Ottawa International Airport
Jock River
SOUTH URBAN COMMUNITY STUDY AREA
Mosquito Creek
RICHMOND
TOWNSHIP OF GLOUCESTER
417
16
MANOTICK
31

REGIONAL MUNICIPALITY OF OTTAWA-CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER

SOUTH URBAN COMMUNITY
REGIONAL SETTING

FIGURE NO. 1

SCALE: 1" = 3 Miles

SEPTEMBER, 1977

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Ottawa River on the north.

The total area involved in this study encompasses approximately 5,400 acres on both sides of the Rideau River, within the political boundaries of the Townships of Nepean and Gloucester, on the west and east respectively.

In addition to the Rideau River, two other water courses drain the study area and carry runoff to the Rideau, namely the Jock River from the west and the Mosquito Creek from the east.

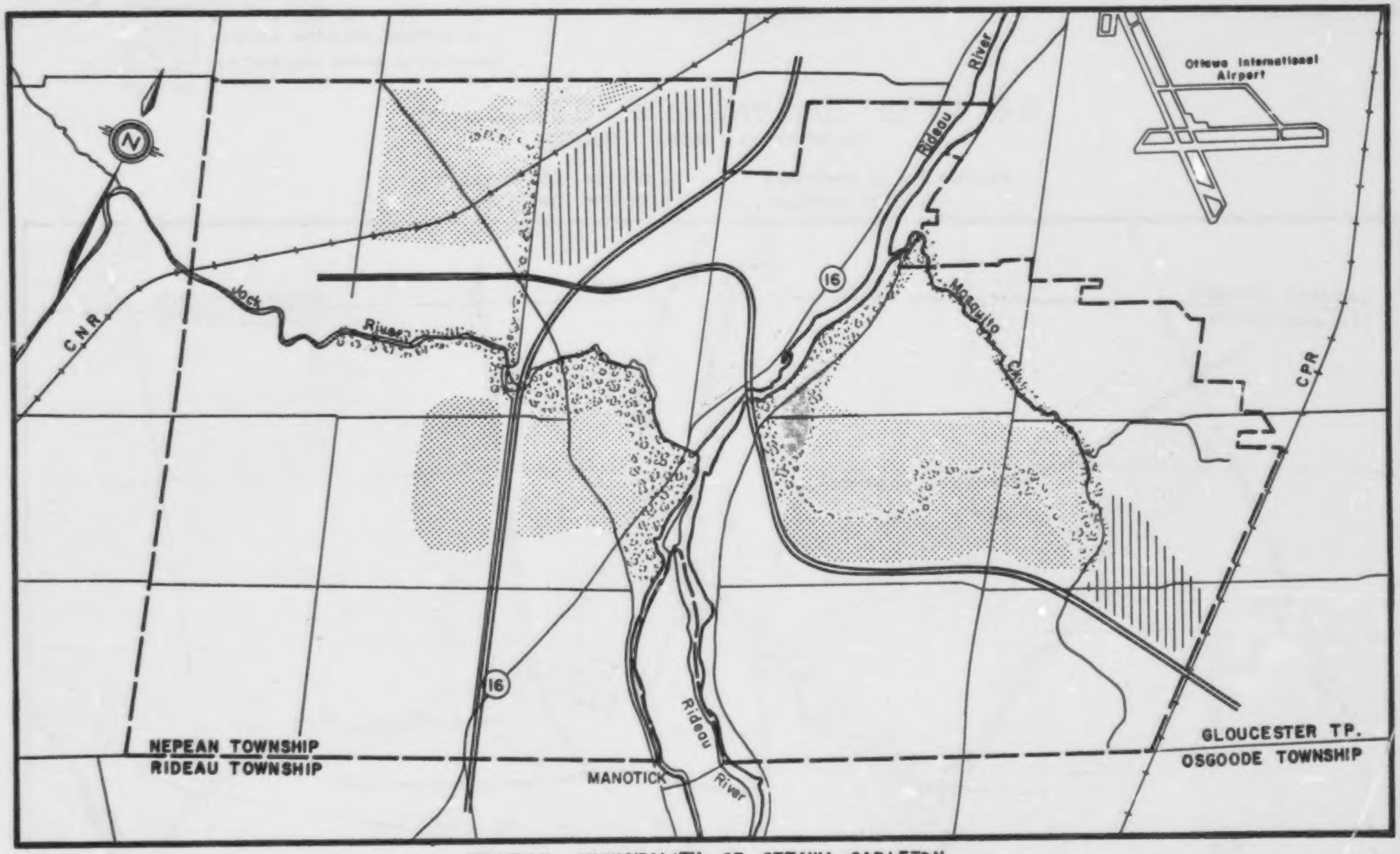
The area is predominantly rural including various agricultural uses extending to the banks of the major water courses. A significant urban area exists in the northwest sector of the study area, while smaller rural development is dispersed over the entire area. Thus, the existing total population is believed to approximate 4,000 people.

PROPOSED DEVELOPMENT

The proposed development is expected to accomodate a population of approximately 100,000 people when completed. In addition to residential development, commercial core and industrial development would also be included. Furthermore, extensive park lands have been proposed, thus taking advantage of the existing waterways and their natural surroundings.

A number of different plans for development have been proposed, however, they differ by small degrees in their layout and extent of the various projected land uses. Figure Nos. 2 & 3 show typical examples of the proposed development plans.

In general, the proposed development aims for a distribution so as to provide for a population of about 65,000 in the Township of Nepean and approximately 35,000 within the lands situated in the Township of


Ottawa International Airport
CNR
CPR
Jock River
Rideau River
Mosquito Crk.
16
16
NEPEAN TOWNSHIP
RIDEAU TOWNSHIP
GLOUCESTER TP.
OSGOODE TOWNSHIP
MANOTICK
N

REGIONAL MUNICIPALITY OF OTTAWA-CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER

SOUTH URBAN COMMUNITY

PROPOSED ALTERNATIVE PLAN 2A

LEGEND:

- ░ Proposed Residential Development
- ▥ Proposed Industrial Development
- ▓ Proposed Core Area
- ▨ Proposed Park Lands

FIG. NO. 2

SCALE: 1" = 5500'
SEPTEMBER 1977

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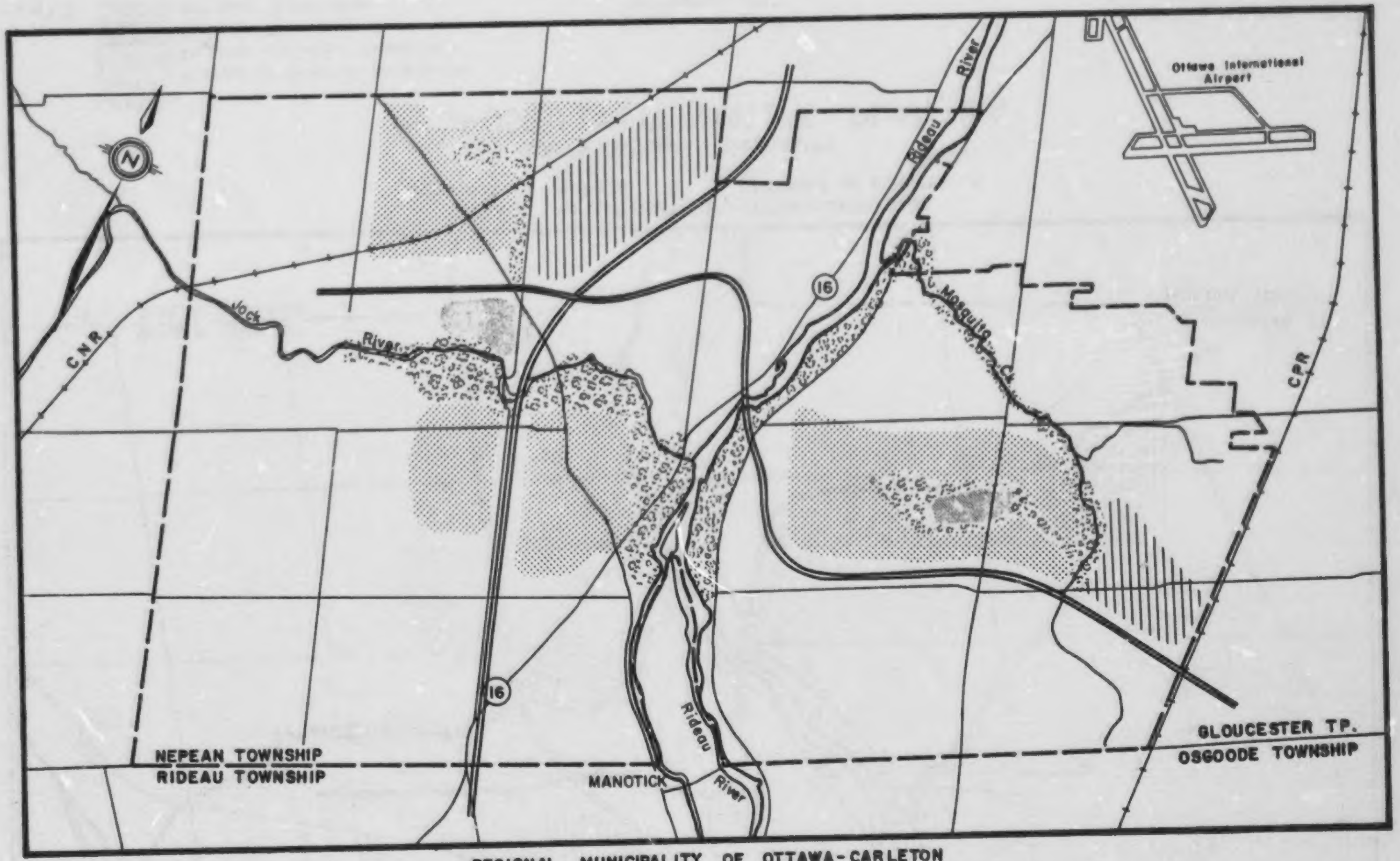
Ottawa International Airport
Rideau River
16
Mosquito Ck.
CPR
CNR
Jock River
16
Rideau River
MANOTICK
NEPEAN TOWNSHIP
RIDEAU TOWNSHIP
GLOUCESTER TP.
OSGOODE TOWNSHIP
N

REGIONAL MUNICIPALITY OF OTTAWA-CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER

SOUTH URBAN COMMUNITY
PROPOSED ALTERNATIVE PLAN 2B

LEGEND:
- ▒ Proposed Residential Development
- ▥ Proposed Industrial Development
- ☐ Proposed Core Area
- ░ Proposed Park Lands

SCALE: 1" = 5500'
SEPTEMBER 1977

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FIG. NO. 3

Gloucester. In addition, industrial development is allocated to portions of the proposed areas in each Township.

RECEIVING WATERS

As mentioned previously, there are three significant watercourses within the designated study area.

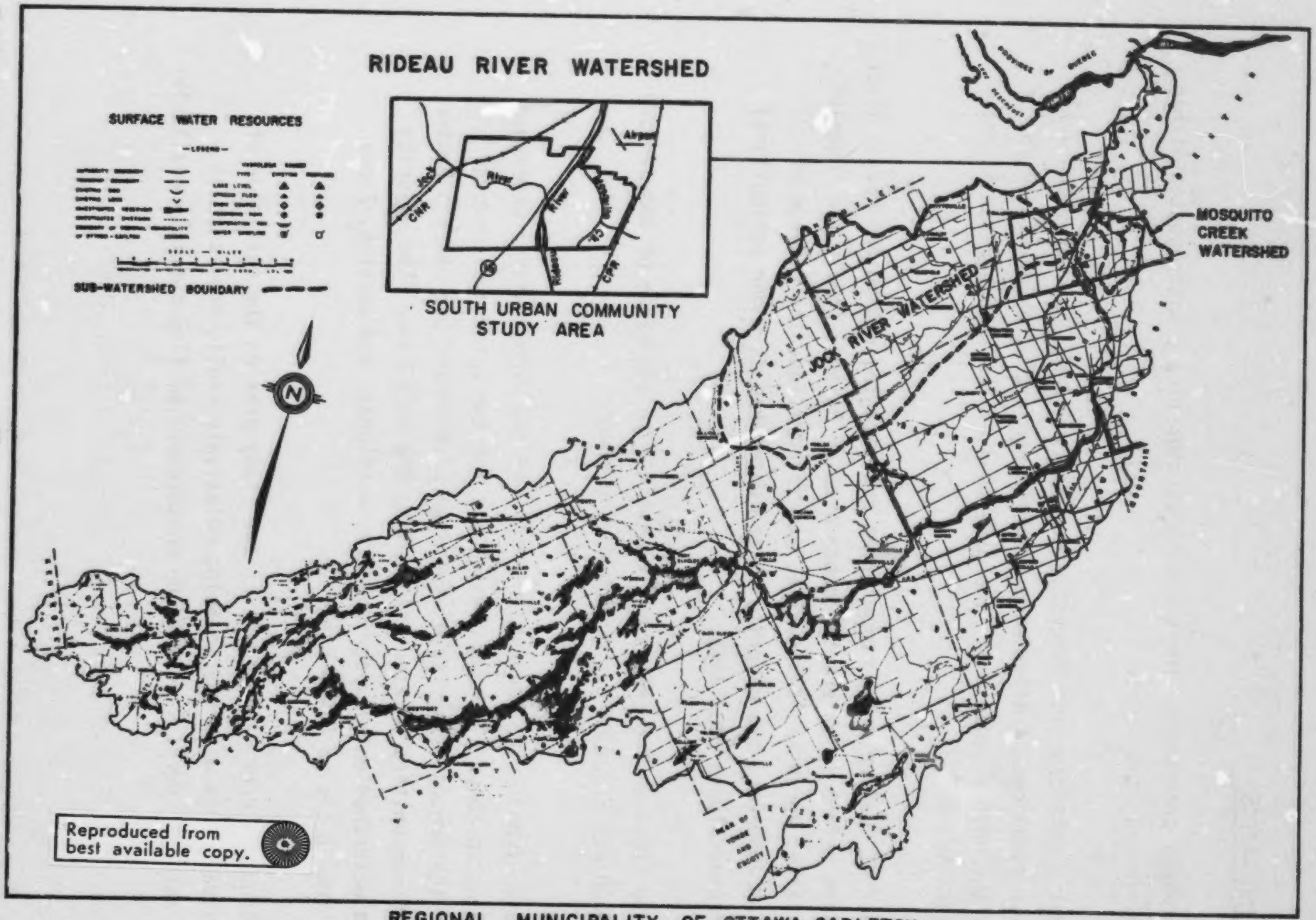
The most significant and largest, the Rideau River, has a drainage area of about 1,480 square miles at its confluence with the Jock River and drains the entire study area. The watershed of the Rideau River is shown on Figure No. 4.

A number of smaller urban communities are situated within the Rideau River watershed before it enters the urban area of Ottawa, namely the Towns of Smith Falls, Perth and Kemptville, and in addition, the Village of Manotick. However, the majority of the watershed is under agricultural use or undeveloped woodlands.

The base flows during the summer months are in the range of 300 cfs and can reach above 20,000 cfs during spring runoff.

The Jock River is the largest tributary of the Rideau with a watershed encompassing about 211 square miles and the two waterways converge within the present study area. The Town of Richmond and Heart's Desire are two communities located adjacent to the Jock River. The majority of the watershed entails agriculture, woodlands, and marshes. Flows range from 2 cfs to 5,000 cfs.

The Mosquito Creek drains most of the study area on the east side of the Rideau. This watercourse being relatively small, drains about 9,700 acres of rural land with no measurement of flow quantity available.


RIDEAU RIVER WATERSHED
SURFACE WATER RESOURCES
— LEGEND —
SUB-WATERSHED BOUNDARY
SOUTH URBAN COMMUNITY STUDY AREA
Jock
River
CNR
Rideau
River
Mosquito Ck.
Airport
CPR
N
MOSQUITO CREEK WATERSHED
JOCK RIVER WATERSHED
PROVINCE OF QUEBEC

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REGIONAL MUNICIPALITY OF OTTAWA-CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER
SOUTH URBAN COMMUNITY
SEPTEMBER 1977

FIGURE NO. 4

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TABLE NO. 1*

(a) SAMPLING RESULTS - RIDEAU RIVER AT JOCK RIVER CONFLUENCE

Date	Time (hrs)	Coliform (MF/100ml)			Dissolved Oxygen (mg/l)	5-Day BOD (mg/l)
		Total	Fecal	Enterococci		
January 5, 1976	10:40	200	10	10	12.0	3.0
February 2, 1976	10:50	400	30	10	11.0	0.4
March 3, 1976	09:50	384	48	60	11.0	1.6
April 12, 1976	13:15	630	1	20	12.0	-
May 3, 1976	10:00	1300	52	32	9.0	1.4
June 1, 1976	09:30	500	-	8	8.0	1.0
July 5, 1976	09:00	400	-	20	10.0	1.8
August 10, 1976	09:30	100	1	12	9.0	3.2
September 7, 1976	10:45	50	2	6	9.0	1.2
October 4, 1976	10:00	200	8	32	9.0	1.2
November 8, 1976	10:00	170	6	4	6.0	0.8
December 6, 1976	09.25	490	10	2	10.0	0.6
Geometric Mean		294	8	12	9.7	1.5
Maximum		1300	52	60	12.0	3.2
Minimum		50	1	2	6.0	0.4

(b) SAMPLING RESULTS - RIDEAU RIVER - 1976

Sampling Location		Coliform (MF/100 ml)			Dissolved Oxygen (mg/l)	5-Day BOD (mg/l)
		Total	Fecal	Enterococci		
Dam at Kilmarnock (55 miles)	Geometric Mean	796	34	8	10.3	1.4
	Maximum	35000	1140	120	12.0	4.6
	Minimum	10	1	0	8.0	0.6
Downstream of Bridge at Kars (24 miles)	Geometric Mean	116	5	5	9.4	1.1
	Maximum	1160	370	30	12.0	1.6
	Minimum	10	1	1	8.0	0.6
Confluence with Jock River - Downstream (15 miles)	Geometric Mean	294	8	12	9.7	1.5
	Maximum	1300	52	60	12.0	3.2
	Minimum	50	1	2	6.0	0.4
Hog's Back Road Ottawa (7.1 miles)	Geometric Mean	369	7	4	10.2	1.0
	Maximum	14800	500	68	12.0	1.2
	Minimum	50	1	1	8.0	0.2
St. Patrick St. Bridge, Ottawa (1.0 miles)	Geometric Mean	298	16	11	11.0	1.1
	Maximum	2900	190	310	18.0	2.1
	Minimum	10	1	1	6.6	0.4

(55 miles) - indicates distance upstream from Ottawa River

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* Ministry of the Environment, Ontario S.E. Region, 1976 River Basins Water Quality Data

WATER QUALITY

Present water quality of the Rideau River indicates a B.O.D. value generally less than 5.0 mg/L and the mean DO to be generally in excess of 6.0 mg/L. Table No. 1 (a) & (b) following, indicates results of sampling undertaken by the M.O.E. on the Rideau River at a location within the area under study as well as at other locations.

The following results are based on limited sampling data but do indicate the general value of quality parameters in the river water. The various quality parameters may range well above the cited values during and following periods of heavy rainfalls.

Some very limited data for the Jock River indicates B.O.D. values generally below 2.0 mg/L and DO levels in the range of 8.0 to 10.0 mg/L, with higher B.O.D. values shown to range from 8.0 to 10.0 mg/L during times of peak runoff.

MINISTRY OF ENVIRONMENT GUIDELINES

Due to the low base flows, the assimilation capacity of the Rideau River during the summer months is limited, and the Ministry view that "further development will have to be based on the principle of no deterioration to the existing quality of local watercourses", the Ministry has formulated a set of guidelines for quality standards of storm water discharge to the Rideau River.

The M.O.E. guidelines provide effluent quality objectives and receiving water quality objectives after mixing. The following water quality objectives refer to average conditions throughout a "one-year" return storm event.

Effluent Objectives

B.O.D.	-	5 to 6 mg/L
Total Phosphorous	-	1 mg/L
Chlorine	-	0.1 mg/L
Total Coliform	-	1000/100 mL
Fecal Coliform	-	100/100 mL
Fecal Streptococci	-	20/100 mL
S.S.	-	maximum removal possible

Receiving Water Criteria

B.O.D.	-	4 mg/L
Chlorine	-	.02 mg/L
Total Coliform	-	1000/100 mL
Fecal Coliform	-	100/100 mL
Fecal Streptococci	-	20/100 mL

STORM WATER MANAGEMENT TECHNIQUES

General

The traditional approach to the problem of urban drainage has been to minimize flooding and inconvenience to the urban community, resulting in efficient storm water collection and transport systems which provide a very rapid discharge to the receiving waters. This in turn has resulted in the following:

a.) rapid washoff and associated increase in contaminants carried to receiving waters,

b.) increased downstream peak flows possibly compounding erosion problems, c.) change in ground cover due to development, with increased imperviousness affecting the natural course of groundwater infiltration, with the possible result of lowering groundwater levels and base flows in natural watercourses.

It is the effect due to increased pollutant loading to the receiving stream, and in this case the Rideau River, which is of prime concern in this study.

Runoff Quality Related to Land Use

The rate of pollutant build-up is commonly associated with land use, as the following table indicates.

TABLE NO. 2

POLLUTANT BUILD-UP RELATED TO URBAN LAND USE

LAND USE	S.S.	B.O.D.	PO_4	N
	(values represent Lbs./Acre per day)			
Residential	4.8	.20	.0022	.02
Commercial	33.6	1.52	.014	.08
Industrial	9.8	.44	.0044	.06

The results in Table No. 2 indicate that the commercial portion of developed land has the highest rate of build-up, followed by industrial and then residential land uses. The amount of contaminants actually washed off depends on the effectiveness of street cleaning practices. Comparison of results in Table Nos. 2 & 3 show the notable effect of street sweeping on commercial land use.

TABLE NO. 3

POLLUTANT WASHOFF FROM URBAN LAND USE

LAND USE	S.S.	B.O.D.	PO_4	N
	(values represent Lbs./Acre per day)			
Residential	4.27	.110	.0019	.018
Commercial	2.58	.066	.0011	.006
Industrial	7.70	.205	.0035	.049

The curves in Figure No. 5 show an idealized version of Quality-Quantity hydrographs during the course of a storm for a typical, medium-sized city.

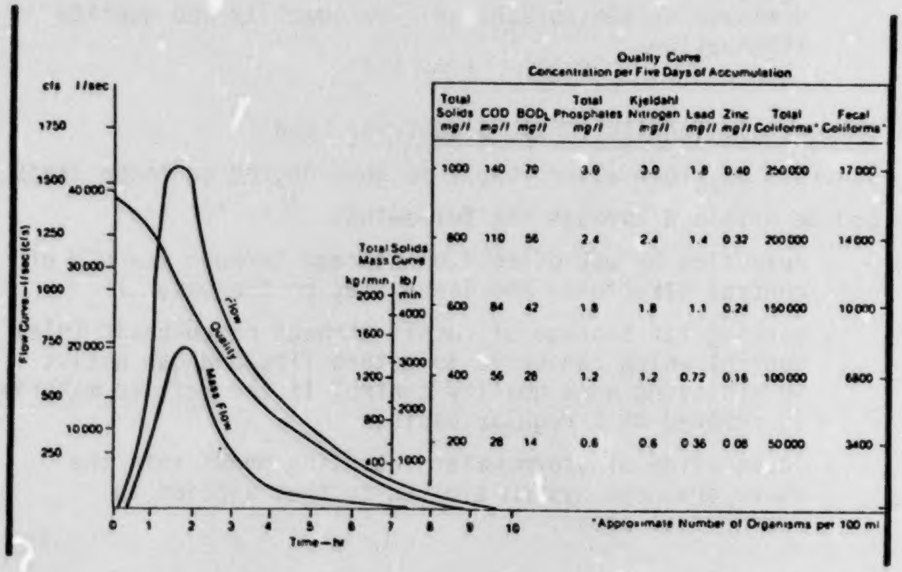


FIGURE NO. 5

TYPICAL QUALITY-QUANTITY HYDROGRAPH

The previous figure shows, in part, that the mass flow curve coincides with the peak flow curve, thus indicating the major portion of total solids to be washed off during the peak of the hydrograph. Conversely, the latter portion of the hydrograph contributes a relatively small portion of the mass loadings to the receiving stream.

Storm Water Runoff Controls

a.) Residential Land

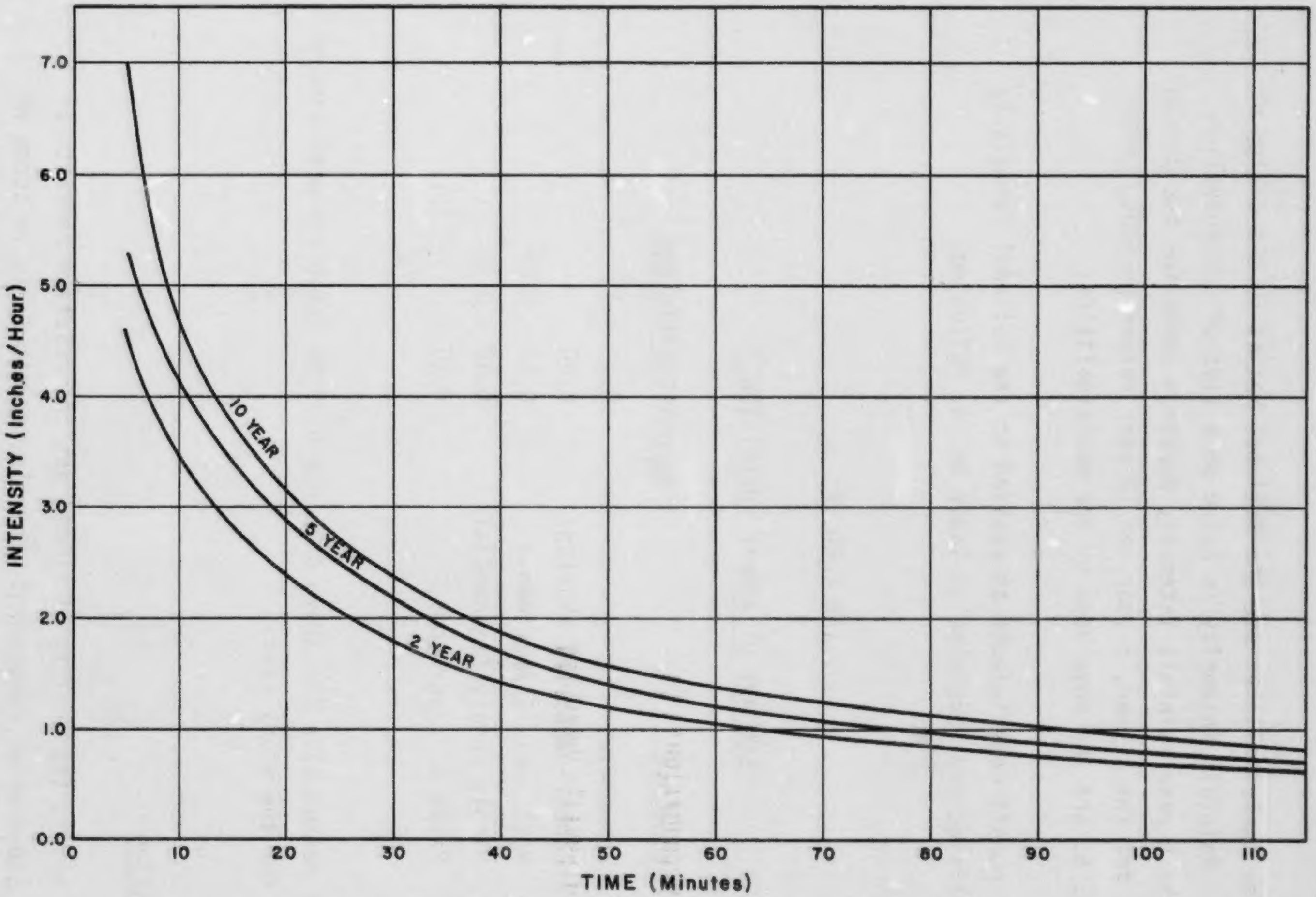
Control of storm water on residential lands can be achieved through:

- minimum grades on residential lots and drainage swales,
- attenuation of peak flows by applying the blue-green concept and directing runoff from impervious areas onto green open space areas,
- improving effectiveness of catch basin trap efficiency through properly maintained sumps or use of filter bags directly under the grating,
- integrating storm water retention ponds into the storm drainage system to achieve flow quantity and quality attenuation.

b.) Commercial, Institutional, Industrial Land

Controls of storm water runoff to some degree on these lands can be attained through the following:

- retention by way of rooftop storage through the use of control structures and large flat roof areas,
- parking lot storage of runoff through catch basin inlet control which can serve to retard flow and can assist in achieving some quality control if the settled material is removed on a regular basis,
- integration of storm water retention ponds into the storm drainage system similar to that applied in residential areas.



REGIONAL MUNICIPALITY OF OTTAWA-CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER

SOUTH URBAN COMMUNITY

RAINFALL INTENSITY DURATION CURVES

FIGURE NO. 6

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SYSTEM DESIGN AND ANALYSIS

Current Design Practice

Presently, the municipalities use the Rational Method in the design of storm sewers. Rainfall intensity is based on a time of concentration applied to the 5-year rainfall intensity duration curve for the Ottawa area. Curves for the 2-year, 5-year and 10-year return periods, shown in Figure No. 6, are the ones used by the municipalities.

A summary of runoff coefficients as applied to the Rational Formula by the municipalities are tabulated in Table No. 4, following:

TABLE NO. 4

SUMMARY OF RUNOFF COEFFICIENTS

Land Use	Runoff Coefficient
Commercial	0.70
Light Industrial	0.60
Multi-Unit & Apartments	0.50 - 0.60
Single Family Residential	0.30 - 0.40
Parks & Playgrounds	0.20

The Rational Method with the above data was used to establish preliminary sewer sizes for the study area.

Model Simulation

In order to assess the quality of runoff and the relative benefits of the various storm water management controls, particularly in terms of storage, it was necessary to apply modelling techniques in this analysis. The models applied and deemed appropriate to this study were the STORM and SWMM.

Significant Quality Parameters Modelled

Parameters of concern related to storm water runoff from rural and urban areas are related to the various functions of the receiving waters.

a.) Public Health

Standards used to measure the safety of a water body for primary recreation uses, such as swimming, relate to levels of total and fecal coliform and fecal streptococcus. The STORM Model version used did not simulate coliform bacteria levels. Although newer versions of the model have this capacity, it is questionable as to how accurate any prediction of this kind would be, given the site specific nature and tremendous variations of these parameters.

b.) Stream Eutrophication

Parameters which are the main source of accelerating the aging process of a water body are Nitrogen and Phosphorous. In light of the M.O.E. guidelines cited earlier, Phosphorous is regarded in this study as the more critical with respect to storm water runoff and therefore has been chosen as a parameter for modelling purposes.

c.) Fish Life

The quality of fish life which can be supported by a water body is greatly dependent on the available oxygen levels. As a result, the Biochemical Oxygen Demand (B.O.D.) contributed by storm water runoff, is chosen as a parameter for simulation.

d.) Suspended Solids

With regard to storm water runoff, the suspended solids content is significant because most other parameters are associated with the solids content. Although the M.O.E. guidelines do not state specific levels for suspended solids in runoff, the simulation of this parameter is most desirable as it will assist in determining the other parameters associated with it. Furthermore, any removal of the solids content from storm water runoff will similarly affect the associated parameters.

Accuracy of Predications

The accuracy of any predictions provided by the models are directly dependent on the reliability of the input data. An intensive literature review was undertaken to arrive at satisfactory pollutant loading rates. For the most part, research has dealt with urban land uses with a more limited emphasis on non-urban, undeveloped lands. As it was desirable to compare the undeveloped and developed state of the study area, both urban and non-urban loading rates were necessary. The most significant sources of pollutant loading information, along with the values suggested by them, are listed in Table No. 5.

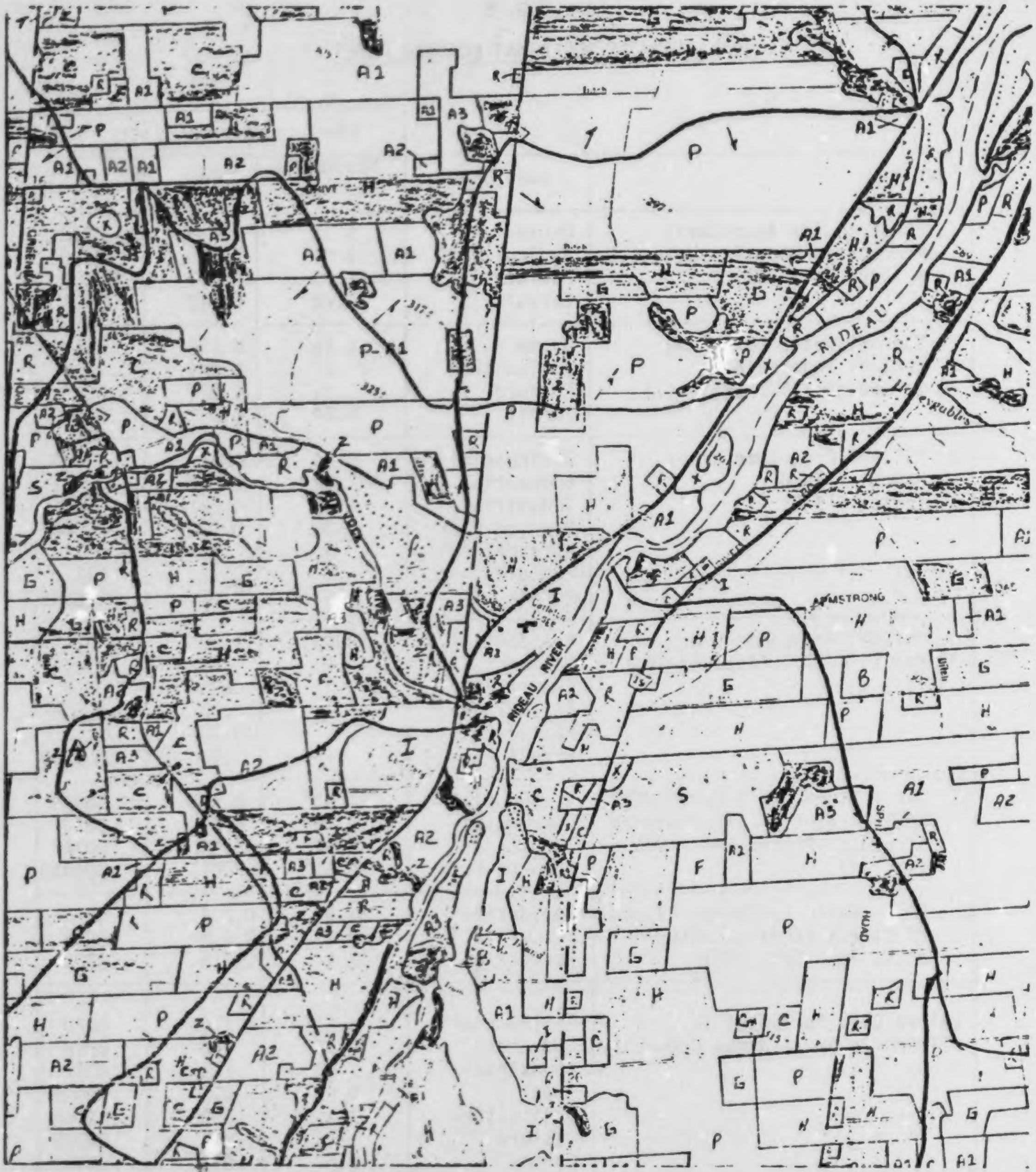
At the bottom of Table No. 5 are shown the actual values used as input for the simulation in this study. These values are similar to Source No. 1 with some adjustment in order to depict a further breakdown of land uses in the urban category. While Source No. 1 was regarded as one of the most comprehensive in this regard, information from the other sources was used judiciously for some modification.

In order to adequately assess the storm water runoff effects from the study area in its present relatively undeveloped state, it was necessary to delineate the various forms of current, predominantly rural land uses. To this end, the Canadian Department of Agriculture was helpful in providing a very detailed log of present land uses

TABLE NO. 5

COMPARISON OF POLLUTANT LOADING RATES

		Loading Rate (lbs/acre/day)		
Source	Land Use	Suspended Solids	BOD.	P
1. EPA-Areawide Assessment Procedures Manual - Volume I - 1976 (Pg. 2-66)	Urban	5.31	0.109	.0020
	Agriculture	3.91	0.063	.0016
	Pasture	1.05	0.027	.0008
	Forest	0.63	0.013	.0004
2. Journal Water Pollution Control Federation - Volume 49, 1977 (Pg. 443)	Urban	4.16	0.183	.0049
	Agriculture	10.27	0.044	.0026
	Pasture	2.05	0.027	.0007
	Forest	0.24	0.012	.0002
3. "STORM" Programme Users Manual - 1974 (Notes 1 & 2)	Residential	0.24	0.011	.00003
	Commercial	1.68	0.076	.00022
	Industrial	0.49	0.022	.00006
	Open or Park	0.05	0.002	.000006
	Agriculture	-	.02	.022
	Pasture	-	3.10	.112
	Forest	-	0.01	.000006
4. "STORM" Programme Users Manual - 1976 (Appendix C)	Residential	0.45	.07	.0020
	Commercial	-	.46	.0127
	Industrial	-	.39	.0096
	Agriculture	-	.02	.00006
	Pasture	-	3.10	.1118
	Forest	-	.01	.000008
5. EPA - STORM WATER MANAGEMENT MODEL: Level I - Preliminary Screening Procedures, 1976 (Pg. 17)	Residential	1.33	0.065	.00086
	Commercial	1.46	0.210	.0016
	Industrial	1.91	0.080	.00147
	Open or Park	0.033	.0013	.000038
6. EPA - Water Pollution Aspects of Street Surface Contaminants, 1972 (Pg. 145)	Residential	33.5	0.204	.0217
	Commercial	10.2	0.056	.0056
	Industrial	42.4	0.218	.0333
Values used for input in "STORM" in South Urban Community Study	Residential	4.27	.110	.0020
	Commercial	2.58	.066	.0012
	Industrial	7.70	.205	.0037
	Open	0.05	.013	.00024
	Agriculture	3.9	.063	.00156
	Pasture	1.05	.027	.00078
	Forest	0.63	.013	.00039



LEGEND OF LAND USE

- P – PASTURE
- H – HAY
- A – AGRICULTURAL (IDLE)
- S – SOD FARMS, NURSERIES
- C – CORN
- G – SMALL GRAINS
- R – RESIDENTIAL
- Z – WOODED AREAS
- F – SUMMER FALLOW
- I – INSTITUTIONAL, CEMETERIES

FIGURE NO. 7

SOUTH URBAN COMMUNITY

CANADA DEPARTMENT OF AGRICULTURE
LOG OF PRESENT LAND USE

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CONSULTING ENGINEERS

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within the entire study area. A section of the map providing this information is shown on Figure No. 7.

The input data depicting pollutant loading rates was applied to the study area in accordance with the information shown on Figure No. 7. In this manner, assessment of the study area in its present form was determined.

In a similar manner, pollutant loading rates for urban land uses, as shown in Table No. 5, were applied to the study area for the various proposed development patterns.

Assessment of Development Proposals

It has been mentioned previously that a number of different proposals for future development of this area have been conceived, while differing to a minor degree. These differences entail somewhat more extensive development areas for some and possible alternate location of the commercial core for others.

In assessing all the development proposals, the STORM Model was used in order to give relative amounts of washoff from all watersheds within the study area.

Table No. 6, following, shows the results in terms of pollutant runoff loading in lbs/acre annually based on a rainfall record of 17 years. The total tributary area used for each development proposal as well as the existing land use was the same so that comparison could readily be made.

It is noted at the bottom of Table No. 6 that the average washoff for the existing case is significantly lower than that for the proposed developments. However, among the 8 differing development plans, there is no great variation in the results shown in this table, although

TABLE NO. 6

SOUTH URBAN COMMUNITY
RESULTANT POLLUTANT RUNOFF LOADINGS
FOR ALTERNATIVE DEVELOPMENT PLAN PROPOSALS*

(lbs./acre annually)

WATERSHED	Existing			Alternative 1A			Alternative 1B			Alternative 2A			Alternative 2B			Alternative 3A			Alternative 3B			Alternative 4A			Alternative 4B		
	S.S.	B.O.D	P	S.S.	B.O.D.	P	S.S.	B.O.D.	P	S.S.	B.O.D	P	S.S.	B.O.D	P	S.S.	B.O.D.	P	S.S.	B.O.D	P	S.S.	B.O.D	P	S.S.	B.O.D.	P
Mosquito-F	80	2	.03	597	99	3.4	384	56	1.9	386	58	2.0	375	56	1.9	372	53	1.8	391	54	1.9	372	55	1.9	388	58	2.0
Mosquito-S	220	4	.1	346	53	1.8	378	62	2.2	359	56	2.0	363	55	1.9	368	55	1.9	346	50	1.7	358	51	1.7	370	55	1.9
Rideau-ES	211	11	.33	218	13	.4	224	14	.4	220	24	.8	233	28	.9	384	55	1.8	365	60	2.0	375	62	2.1	446	72	2.5
Rideau-EN	185	23	.69	185	23	.7	185	23	.7	219	29	.9	185	23	.7	228	31	1.0	185	23	.7	211	32	1.1	185	23	.7
Barrhaven-N	169	13	.38	741	121	4.2	749	123	4.3	713	118	4.1	717	118	4.1	695	114	3.9	700	115	4.0	726	120	4.1	724	119	4.1
Barrhaven-S	254	4	.11	382	53	1.8	310	51	1.8	356	46	1.6	370	53	1.8	332	40	1.3	372	54	1.9	428	64	2.2	450	68	2.3
Hearts Desire	319	42	1.27	319	42	1.3	442	65	2.2	319	42	1.3	319	42	1.3	319	42	1.3	466	64	2.1	360	58	1.9	465	64	2.1
Rideau-NW1	41	2	.01	41	2	.01	41	2	.01	366	59	2.0	41	2	.01	256	41	1.3	63	2	.01	41	2	.01	54	2	.01
Jock-SE	266	5	.07	437	66	2.2	431	65	2.2	389	53	1.8	374	49	1.6	365	47	1.5	451	70	2.4	485	79	2.7	504	82	2.8
Jock-SC	250	4	.09	461	73	2.5	440	72	2.5	399	63	2.1	388	60	2.0	345	51	1.7	383	61	2.1	381	57	1.9	424	66	2.2
Jock-SW	224	4	.07	502	82	2.8	494	81	2.8	267	26	.8	316	38	1.3	257	24	.7	249	22	.7	324	40	1.3	324	40	1.3
Rideau-SW1	163	7	.11	504	82	2.8	504	82	2.8	195	26	.9	248	34	1.1	267	31	1.0	258	28	.9	285	34	1.1	237	33	1.1
Rideau-SW2	192	7	.11	506	82	2.8	505	82	2.8	306	35	1.1	332	41	1.3	307	35	1.1	316	38	1.2	233	17	.4	299	33	1.1
Mud Creek-S	198	4	.03	504	82	2.8	503	82	2.8	198	4	.03	198	4	.03	198	4	.03	198	4	.03	198	4	.03	198	4	.03
TOTAL AVERAGE	198	9	.24	410	62	2.11	399	61	2.1	335	46	1.53	319	43	1.42	335	45	1.45	339	46	1.55	341	48	1.60	362	51	1.72

some higher washoff is indicated by Alternatives 1A and 1B. These proposals involve a somewhat greater extent of development within the study area, while allowing for the same total population.

In view of the fact that none of the discussed proposals have been adopted, this analysis with the results indicated in Table No. 6, was considered in making the choice of one development plan proposal for further analysis. As a result Alternative No. 2B along with a sub-option for the location of the central area, has been chosen as being representative of the proposed development pattern. This plan is used for the purpose of developing a storm sewer system and associated storm water management facilities.

APPLICATION OF STORM WATER MANAGEMENT CONTROLS

Street Sweeping

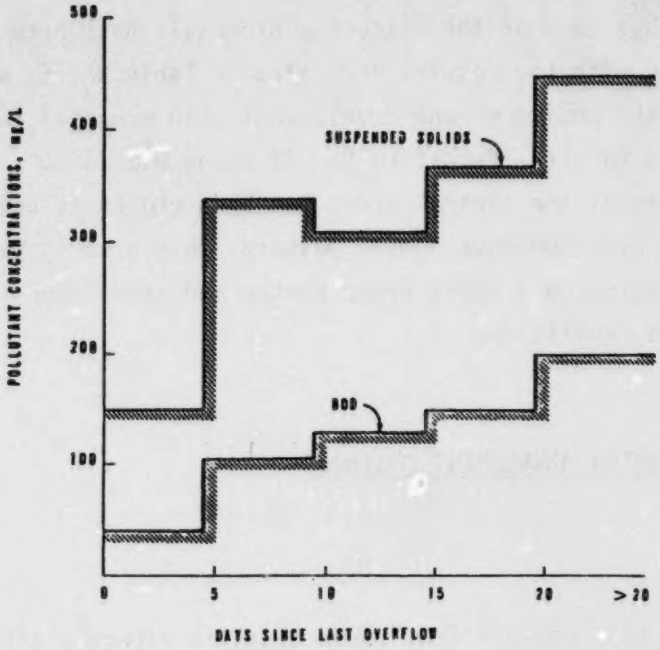
It has been indicated that washoff from urban areas is directly affected by the effectiveness of street sweeping. Figure No. 8 shows the increase in pollutant concentration versus time for combined sewers and urban storm water runoff. These plots imply that an effective street sweeping program needs to be applied within a time interval more frequent than 20 days. As a result, the following street sweeping time intervals are recommended as maximums for the proposed development area.

Residential - 2 weeks
Industrial - 2 weeks
Commercial - 2 to 3 days

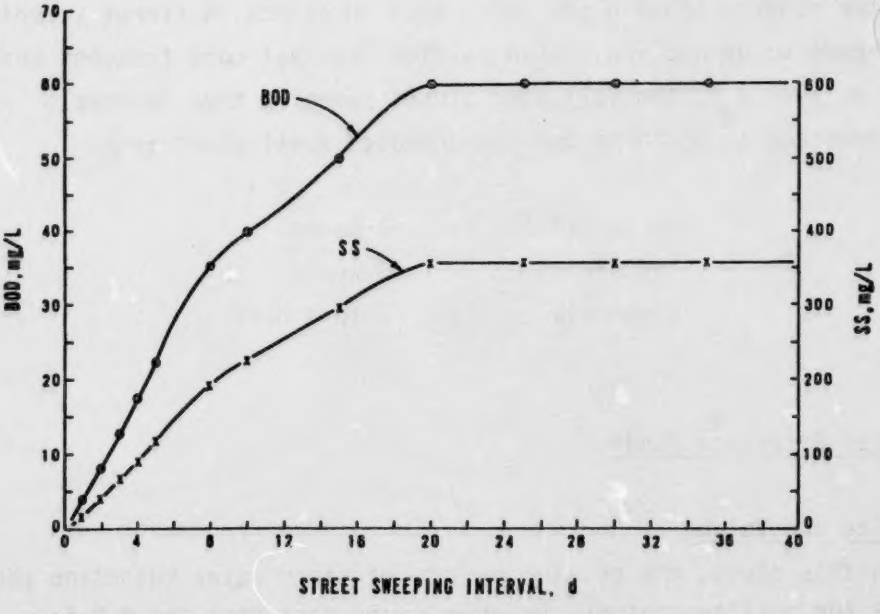
Storm Water Retention Ponds

a.) Size and Volume

In this study, the primary purpose of storm water retention ponds is for quality control. In view of the fact that the M.O.E.

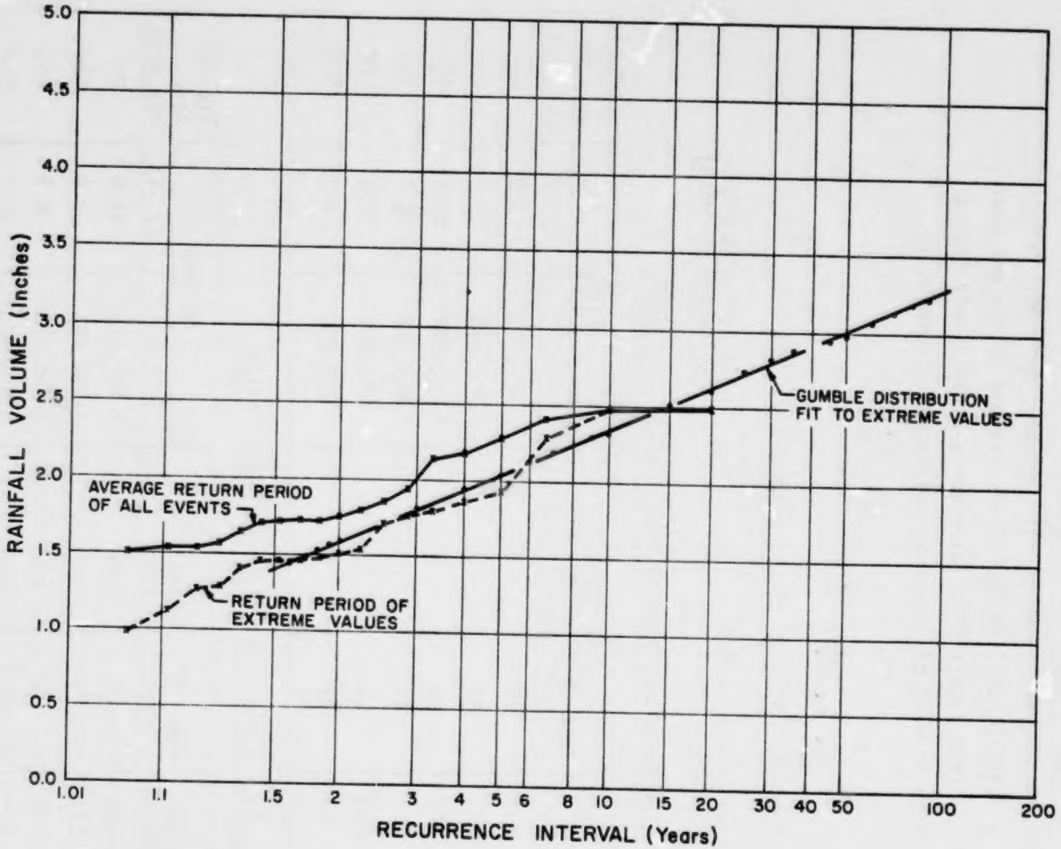


(a) COMBINED SEWER POLLUTANT CONCENTRATION VERSUS PRECEDING DRY-WEATHER PERIOD, MILWAUKEE, WISCONSIN.



(b) STORM WATER POLLUTANT CONCENTRATION VERSUS STREET SWEEPING INTERVAL, DES MOINES, IOWA.

FIGURE NO. 8



REGIONAL MUNICIPALITY OF OTTAWA-CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER

SOUTH URBAN COMMUNITY

PLOT OF RAINFALL VOLUME AND RECURRENCE INTERVAL

FIGURE NO. 9

SEPTEMBER 1977

GORE & STORRIE LIMITED
CONSULTING ENGINEERS

Quality Objectives made reference to a "one year" storm, a statistical analysis was applied to Ottawa rainfall records to establish a one year storm. Analysis of arrays including all events, annual extreme events and a Gumbel Distribution fit to the extreme values, are shown in the plot on Figure No. 9. As a result, a rainfall of 1.50 inches was assumed as the one year storm. The storage required to fully contain the runoff from a one-year storm in each of the urban watersheds, is shown in Table No. 7.

TABLE NO. 7

COMPARISON OF STORAGE VOLUME REQUIREMENTS FOR

OPTIMUM SIZE BASED ON RAINFALL RUNOFF ANALYSIS (STORM) AND 1-YEAR STORM EVENT (1.50 INCHES OF RAIN)

Optimum Size Based on Rainfall-Runoff Analysis (STORM)

Watershed	Area (acres)	Avg. "C" ("STORM")	No. of Bypasses (Yr.)	Volume Ft.3	Volume Ac.-Ft.	Storage Acre-Ft. per Acre
Mosquito-Ind.	405	.49	3	480,769	11.0	.027
Gloucest-Res.	1521	.40	3	1,442,308	33.1	.022
Barrhaven-N	1148	.46	3	1,282,051	29.4	.026
Barrhaven-S	815	.46	3	961,538	22.1	.027
Jock-South	833	.39	3	801,282	18.4	.022
Rideau-SW	323	.37	3	272,436	6.25	.019

1 Year Storm Event (1.50 inches of rain)

Watershed	Area (acres)	Avg. "C" ("STORM")	Runoff (in.)	Volume Ft.3	Volume Ac.-Ft.	Storage Acre-Ft. per Acre
Mosquito-Ind.	405	.49	.740	1,087,910	25.0	.062
Gloucest-Res.	1521	.40	.640	3,334,821	76.6	.050
Barrhaven-N	1148	.46	.695	2,896,228	66.5	.058
Barrhaven-S	815	.46	.695	2,056,120	47.2	.058
Jock-South	833	.39	.589	1,781,011	40.9	.049
Rideau-SW	323	.37	.559	655,421	15.0	.046

A second method was used to arrive at the size of the storm water retention facilities. The STORM Model was applied to analyse the actual rainfall record over an extended period, with the land use as indicated on proposed development plans and a pre-set flow through rate based on a desired detention time. A plot indicating the results of this analysis for the 6 urban watersheds is shown on Figure No.10. Plots in Figure No.10 indicate an optimum storage size to be where the curves cease to provide a significant decrease in overflows for increased storage volume. Therefore, a storage volume corresponding to about 3 bypasses per year was chosen as the optimum.

Table No. 7 shows the required storage volumes for each urban watershed, based on the one year storm volume and on the STORM Model analysis. Comparison of the data in the table shows a volume requirement for the one year storm to be more than double that shown as the optimum by the STORM analysis. Therefore, the optimum storage volumes based on the latter method were used to establish the size of the storm water retention ponds.

b.) Location

The location of the various ponding facilities was determined based on providing a minimal number of outlets and incorporating the storage facilities into the proposed trunk sewer systems. It was desirable to have a ponding facility at the downstream end of each storm sewer drainage system, in the case that some form of treatment such as chemical addition could be readily applied if the need arose at some time in the future. The locations of all necessary ponding facilities have been shown on the Storm Drainage Master Plan in Figure No. 12.

Storage facilities have already been planned or constructed for portions of the study area located in Nepean Township, north of the Jock River, and have been shown accordingly. Others have been indicated in accordance with the analysis just described.

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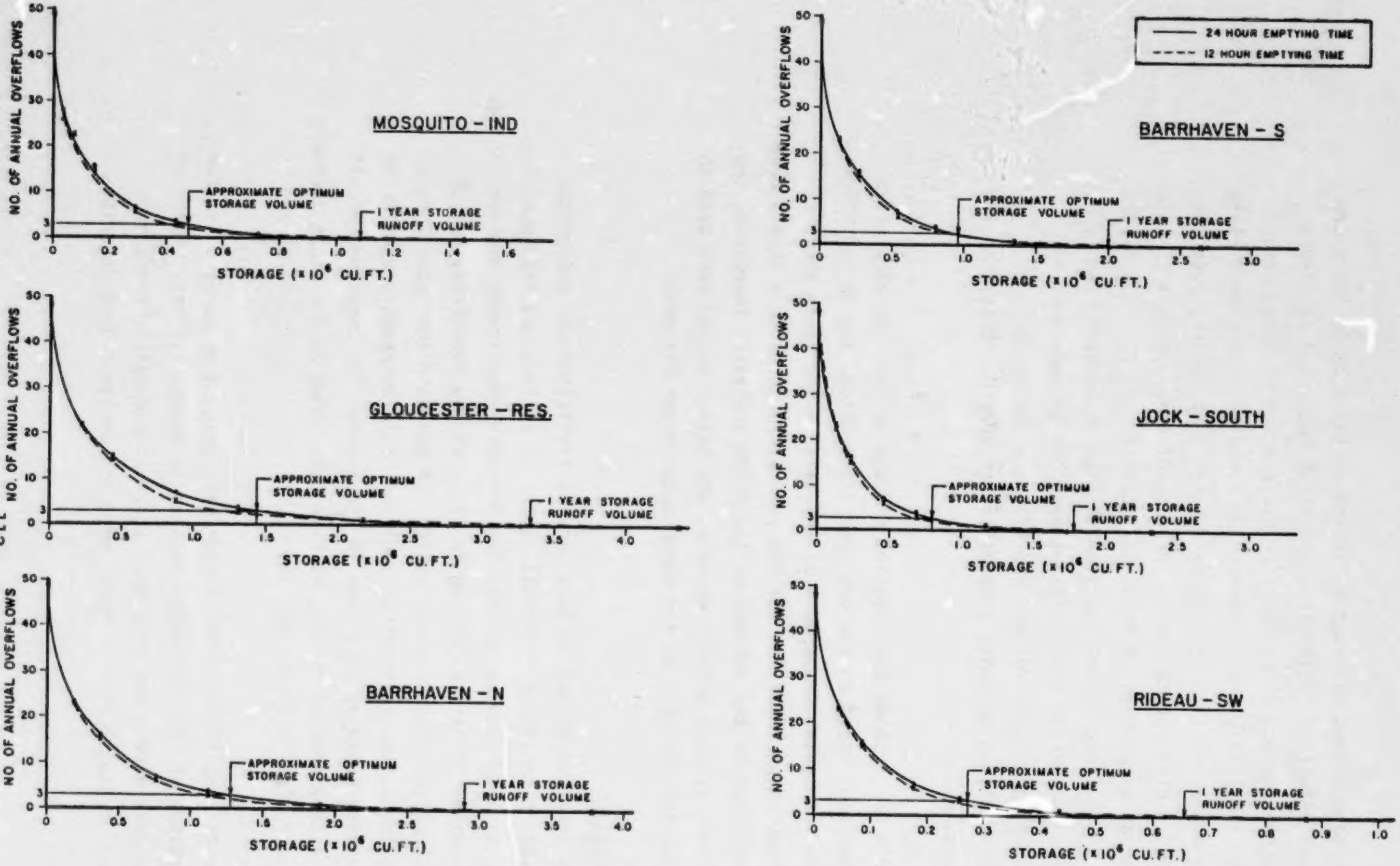

REGIONAL MUNICIPALITY OF OTTAWA - CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER

SOUTH URBAN COMMUNITY

PLOT OF ANNUAL NO. OF OVERFLOWS VERSUS STORAGE CAPACITY

FIGURE NO. 10

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With regard to the watersheds in Gloucester Township, consideration was given to the apparent erosion problems along the Mosquito Creek. As a result, outlets to this watercourse were minimal, with the major portion of the runoff carried as far downstream as possible.

Alternative locations for the commercial core area have also been included in the analysis and are also shown on Figure No. 12.

Quality of Runoff

a.) Continuous Simulation

It was desirable to obtain average annual values for quantity and quality of runoff being generated by the proposed development area. The STORM Model was used to predict these values based on recommended storage provided and pre-set flow through rates.

The effectiveness of the retention facilities, based on available literature and ongoing research projects, has been assumed as follows, for a retention period of 24 hours:

Suspended Solids Removal - 90%
B.O.D. Removal - 80%
Phosphorous Removal - 60%

In addition, preliminary results of current research indicate a 24-hour retention time to result in an effective die off rate of coliform bacteria.

Table Nos. 8 & 9 summarize the quality of runoff entering the storm water retention ponds, the effectiveness of the ponds and the quality of discharge to the receiving waters. The values in these tables indicate that the quality of discharge to the receiving waters closely approaches the previously noted guidelines of the M.O.E.

TABLE NO. 8

TOWNSHIP OF NEPEAN

SUMMARY OF AVERAGE ANNUAL WASHOFF AND STORAGE TREATMENT EFFECTIVENESS

	Watershed	Volume of Washoff (ft^3)	Total Pollutants In Washoff (lbs)			Concentration of Pollutant (MG/L)		
			SS	BOD	P	SS	BOD	P
No Storage or Treatment	Barrhaven-N	21,086,234	854,107	83,561	4,878	649	64	3.7
	Barrhaven-S	14,910,588	402,539	38,644	2,302	433	42	2.5
	Jock-South	12,790,632	373,766	36,200	2,136	468	45	2.7
	Rideau-SW	4,760,310	133,883	13,055	758	451	44	2.6
		53,547,764	1,764,295	171,460	10,074	528	51	3.0
Flow Into Ponds	Barrhaven-N	20,377,803	775,887	75,908	4,431	610	60	3.5
	Barrhaven-S	14,910,588	402,539	38,644	2,302	433	42	2.5
	Jock-South	12,730,156	371,999	36,029	2,126	468	45	2.7
	Rideau-SW	4,760,310	133,883	13,055	758	451	44	2.6
		52,778,857	1,684,308	163,636	9,617	511	50	2.9
Flow From Ponds	Barrhaven-N	20,377,803	77,589	15,182	1,772	61	12	1.4
	Barrhaven-S	14,910,588	40,254	7,729	921	43	8	1.0
	Jock-South	12,731,156	37,200	7,206	850	47	9	1.1
	Rideau-SW	4,760,310	13,388	2,611	303	45	9	1.0
		52,778,857	168,431	32,728	3,846	51	10	1.2
Flow Bypassed	Barrhaven-N	708,431	26,974	2,639	154	610	60	3.5
	Barrhaven-S	-	-	-	-	-	-	-
	Jock-South	60,476	1,767	171	10	468	45	2.6
	Rideau-SW	-	-	-	-	-	-	-
		768,907	28,741	2,810	164	599	59	3.4
Total Loading to Receiving Water	Barrhaven-N	21,086,234	104,563	17,821	1,926	79	14	1.5
	Barrhaven-S	14,910,588	40,254	7,729	921	43	8	1.0
	Jock-South	12,790,632	38,967	7,377	860	49	9	1.1
	Rideau-SW	4,760,310	13,388	2,611	303	45	9	1.0
		53,547,764	197,172	35,538	4,010	59	11	1.2

TABLE NO. 9

TOWNSHIP OF GLOUCESTER

SUMMARY OF AVERAGE ANNUAL WASHOFF AND STORAGE TREATMENT EFFECTIVENESS

	Watershed	Volume of Washoff (ft^3)	Total Pollutants In Washoff (lbs)			Concentration of Pollutant (MG/L)		
			SS	BOD	P	SS	BOD	P
No Storage or Treatment	Mosquito-Ind.	7,953,510	365,094	35,826	2,094	736	72	4.2
	Gloucester-Res.	23,244,378	631,094	61,802	3,599	435	43	2.5
		31,197,888	996,188	97,628	5,693	512	50	2.9
Flow Into Ponds	Mosquito-Ind.	7,894,704	326,155	32,005	1,871	662	65	3.8
	Gloucester-Res.	23,023,529	625,097	61,215	3,565	435	43	2.5
		30,918,233	951,252	93,220	5,436	493	48	2.8
Flow From Ponds	Mosquito-Ind.	7,894,704	32,616	6,401	748	66	13	1.5
	Gloucester-Ind	23,023,529	62,510	12,243	1,426	44	9	.99
		30,918,233	95,126	18,644	2,174	49	10	1.11
Flow Bypassed	Mosquito-Ind.	58,805	2,429	238	14	662	65	3.8
	Gloucester-Res.	220,849	5,996	587	34	435	43	2.5
		279,654	8,425	825	48	483	47	2.8
Total Loading To Receiving Water	Mosquito-Ind.	7,953,510	35,045	6,639	762	71	13	1.5
	Gloucester-Res.	23,244,378	68,506	12,830	1,460	47	9	1.0
		31,197,888	103,551	19,469	2,222	53	10	1.1

TABLE NO. 10

COMPARISON OF STORM WATER RUNOFF CHARACTERISTICS
BEFORE AND AFTER URBANIZATION

TOWNSHIP OF NEPEAN

BEFORE URBANIZATION

AFTER URBANIZATION

WATERSHED					BEFORE PONDING				AFTER PONDING			
		Total Pollutant Washoff (lbs/yr)				Total Pollutant Washoff (lbs/yr)						
	Volume of Washoff (ft^3)	S.S.	BOD	P	Volume of Washoff (ft^3)	S.S.	BOD	P	Volume Washoff (ft^3)	S.S.	BOD	P
Barrhaven - N	5,250,722	194,012	14,924	436	21,086,234	854,107	83,561	4,878	21,086,234	104,563	17,821	1,926
Barrhaven - S	2,544,267	207,010	3,260	90	14,910,588	402,539	38,644	2,302	14,910,588	40,254	7,729	921
Jock - South	2,630,697	208,250	3,332	75	12,790,632	373,766	36,200	2,136	12,790,632	38,967	7,377	860
Rideau - S.W.	1,031,791	85,918	1,615	23	4,760,310	133,833	13,055	758	4,760,310	13,388	2,611	303
TOTAL	11,457,477	695,190	23,131	624	53,547,764	1,764,295	171,460	10,074	53,547,764	197,172	35,538	4.010

TOWNSHIP OF GLOUCESTER

BEFORE URBANIZATION

AFTER URBANIZATION

WATERSHED					BEFORE PONDING				AFTER PONDING			
		Total Pollutant Washoff (lbs/yr)				Total Pollutant Washoff (lbs/yr)						
	Volume of Washoff (ft^3)	S.S.	BOD	P	Volume of Washoff (ft^3)	S.S.	BOD	P	Volume of Washoff (ft^3)	S.S.	BOD	P
Mosquito - Ind.	1,129,031	32,227	698	17	7,953,510	365,094	35,826	2,094	7,953,510	35,045	6,639	762
Gloucester - Res.	5,410,805	327,015	11,408	327	23,244,378	631,094	61,802	3,599	23,244,378	68,506	12,830	1,460
TOTAL	6,689,836	359,242	12,106	344	31,197,888	996,188	97,628	5,693	31,197,888	103,551	19,469	2,222

Table No. 10 shows the results of the comparison made on the various runoff parameters for the study area under conditions of present land use and after urbanization. This table clearly shows the increase in the various runoff characteristics due to urbanization. However, it is also shown that the implementation of the storm water management practices, particularly the retention facilities, can reduce significantly the amount of pollutant washoff.

b.) Instantaneous Simulation

The SWMM Model was used to model a single event with a one year synthetic storm based on a hyetograph developed by Keifer and Chu.

Characteristics dealing with the watershed and used in the SWMM simulation are identical to those used with STORM, including the recommended street sweeping interval. Figure No. 11 shows the resulting inflow and outflow parameters for 2 urban watersheds, based on results from the SWMM Model.

DESIGN OF STORM WATER DRAINAGE SYSTEM

Initially trunk storm sewer sizes were designed using the Rational Method based on a storm with a 5-year return period, as described previously.

The retention pond volume was established based on a limited number of overflows in an average year and are primarily used for quality control. However, the ponding facilities will assist in attenuating peak flows due to added storage being available.

The design of retention ponds, in addition to the total volume requirements, should include the following:

a.) normal water depth 4 to 6 feet,

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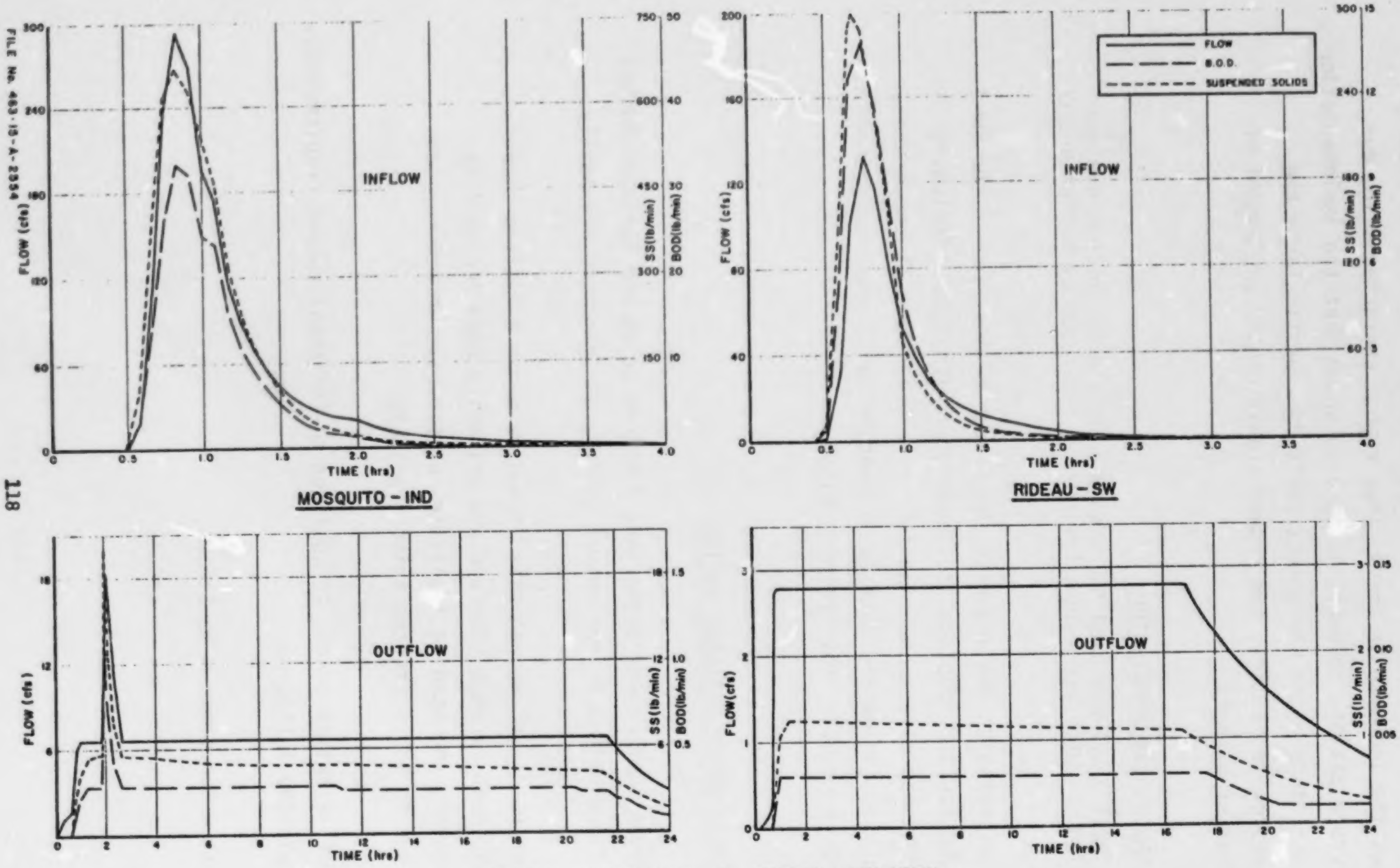


REGIONAL MUNICIPALITY OF OTTAWA - CARLETON
TOWNSHIP OF NEPEAN TOWNSHIP OF GLOUCESTER

SOUTH URBAN COMMUNITY
RETENTION POND HYDROGRAPHS AND POLLUTOGRAPHS

GORE & STORRIE LIMITED
CONSULTING ENGINEERS

FIGURE NO. 11

b.) provision of emergency overflow or bypass facilities in the case of a severe storm exceeding the design volume,

c.) minimum pond area of 5 acres for aesthetic and recreational reasons,

d.) side slopes of banks should be minimum 7 horizontal to 1 vertical for safety and ease of maintenance purposes,

e.) submergence and marking of inlet and outlet pipes for safety purposes,

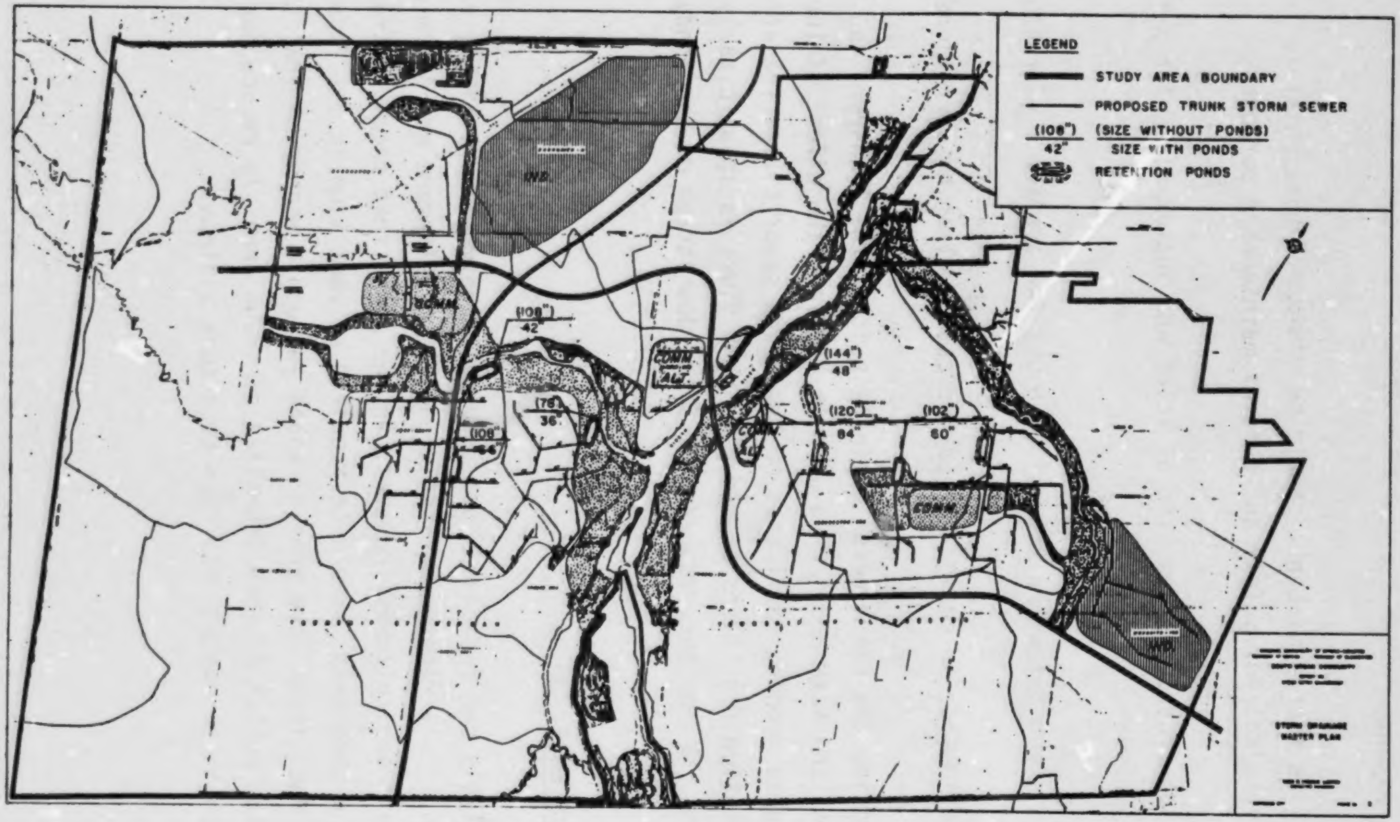
f.) minimum detention time of 24 hours for average 1 year storm,

g.) allowance in design for any necessary cleaning operations.

In designing the storm water drainage system, the Transport and Storage Blocks of the SWMM Model were used to simulate the effect of the recommended ponding facilities on the trunk sewer sizes. The effect of the retention facilities on reducing peak flows is shown clearly in Figure No. 12 , by the reduced pipe sizes downstream of the ponds.

Treatment of Storm Water

The analysis has indicated that chemical treatment will likely not be required if all recommendations regarding development and storm water management are followed. However, in the event that chemical treatment such as a coagulant should be required at some time in the future, the ponding facilities have been arranged in such a manner so that there is one at the outlet of each urbanized drainage area. To reduce costs, the number of these outlets has been kept to a minimum.


LEGEND
STUDY AREA BOUNDARY
PROPOSED TRUNK STORM SEWER
(108") / 42"
(SIZE WITHOUT PONDS) / SIZE WITH PONDS
RETENTION PONDS
IND.
COMM.
COMM. ALT.
COMM.
(108") / 42"
(76") / 36"
(108")
(144") / 48"
(120") / 84"
(102") / 60"
COMM.
IND.
STORM DRAINAGE MASTER PLAN

SOUTH URBAN COMMUNITY

FIGURE NO. 12

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STAGING OF CONSTRUCTION AND ESTIMATED COSTS

Staging

The runoff from construction sites is known to transport a significant amount of solids to the receiving watercourse. In view of this, it is recommended that construction of the storm drainage system commence at the downstream end of each drainage area with the retention facility. This will provide some form of control for the removal of solids originating in the active construction areas.

Estimated Costs

Table No. 11 summarizes the storm drainage system costs for the traditional piped system based on the Rational Method Design and also for the controlled system including the storm water management controls described earlier in this study.

The costs shown include the cost of construction of the sewers and ponding facilities plus an allowance of 15% for engineering and contingencies, based on 1977 costs.

The system costs, considering the acreage involved in the proposed development areas, amounts to approximately $8,400 per acre or about $800 above the cost for the solely piped system. This increased cost, however, can provide benefits over and above those available through the traditional storm drainage design.

TABLE NO. 11

ALTERNATIVE SYSTEM DESIGN COST COMPARISON

TOWNSHIP OF NEPEAN
(1156 acres)*

WATERSHED	SYSTEM			
	Uncontrolled Pipe System	Controlled System of Pipes and Ponds		
	Pipe	Pipe	Ponds	Total
Jock - S	6,555,000	5,910,000	1,220,000	7,130,000
Rideau - S.W.	2,255,000	2,015,000	595,000	2,610,000
Total	8,810,000	7,925,000	1,815,000	9,740,000

* Area does not include Barrhaven, which system has already been designed and is to a large extent, constructed.

TOWNSHIP OF GLOUCESTER
(1926 acres)

WATERSHED	SYSTEM			
	Uncontrolled Pipe System	Controlled System of Pipes and Ponds		
	Pipe	Pipe	Ponds	Total
Mosquito - Ind.	2,225,000	2,615,000	595,000	3,210,000
Gloucester - Res.	12,490,000	9,785,000	3,180,000	12,965,000
Total	14,715,000	12,400,000	3,775,000	16,175,000

SUMMARY

The use of modelling techniques has proved to be invaluable in the analysis carried out in this study. The STORM Model was particularly useful for the purpose of determining the relative effects on storm water runoff due to existing land uses and various proposed developments. The SWMM Model enabled the design of the storm drainage system, based on the use of storage facilities.

Although precise calibration of the Models was not feasible during the course of this study, such calibration would be desirable for any application of these models.

With regard to input data and particularly pollutant loading rates, these have been based on the most recent literature available on the subject. However, further research of this kind should clarify precisely the climatological conditions such as annual precipitation, temperature range and thus indicate the actual time interval represented by annual loading rates. Furthermore, pollutant loading rates may be based on "build-up" or "washoff" of pollutants, total or soluble portions, total land area or only impervious portions. These items should be carefully analysed in the application of loading rates to the particular models.

REFERENCES

1. American Public Works Association, "Water Pollution Aspects of Urban Runoff", PB 215 532 (1969)

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METHODOLOGY FOR CALIBRATING
STORMWATER MODELS

By Thomas K. Jewell[1], Thomas J. Nunno[2],
and Donald Dean Adrian[3], M. ASCE

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[1] Instructor, Department of Civil Engineering, University of Massachusetts, Amherst, 01003

[2] Research Assistant, Department of Civil Engineering, University of Massachusetts, 01003

[3] Professor, Department of Civil Engineering, University of Massachusetts, Amherst, 01003.

INTRODUCTION

A primary goal of the Water Pollution Control Act Amendments of 1972 (PL 92-500) is achievement of water quality that allows fishing and swimming in all our nation's waters by 1983. Section 208 of PL 92-500 requires the preparation of areawide waste management plans that define the best mix of point and nonpoint[1] control strategies to meet this goal. Development of viable plans is contingent upon knowing the nature of nonpoint pollutants. In urban areas, nonpoint pollutants are carried by urban stormwater runoff; therefore the characterization of stormwater runoff is of primary importance.

Difficulties involved in gathering storm runoff data and time and monetary constraints make the complete characterization of urban stormwater runoff from each catchment nearly impossible. Therefore, increased emphasis is being placed on the use of computer models for predicting quantity and quality of urban stormwater runoff. These computer models are cost effective and reasonably accurate substitutes for extensive field data gathering programs, provided that the models are calibrated and verified for conditions existent on the particular catchment being studied.

Calibration involves minimization of deviation between measured field conditions and model output by adjusting parameters within the model. Some minimum amount of field data is required to accomplish a reasonable calibration. Verification is the process of checking the model calibration using an independent set of data. Ideally, the verification data set should be as large as the calibration data set. However, if limited data are available, it is usual practice to use the larger portion of the data for calibration and the smaller portion of the data for verification. A poor verification will indicate that more storm events reflecting a greater range of conditions are needed in the calibration data set, or that the model is inadequate and should be modified or rejected.

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[1] Point sources of pollution are defined as the pollutant discharges from municipal sanitary sewer systems and industrial waste discharges. The public or private agency having jurisdiction over point sources of pollutant discharge must obtain a National Pollutant Discharge Elimination System Permit (NPDES permit), under Section 402 of PL 92-500. Nonpoint sources of pollution are all other pollutant discharge sources including agricultural runoff, storm water runoff and combined sewer overflows.

METHODS OF CALIBRATION

Most models for predicting quality of stormwater runoff are coupled with a quantity simulation model. Calibration of these models has consisted of calibration of the quantity and quality models sequentially, using the output from the calibrated quantity model as input to the quality model (5) Sometimes only the quantity portion has been calibrated and default or estimated values have been used for quality parameters (14).

In talking with practitioners of the art of stormwater management modeling, it has become apparent that there are nearly as many ways to calibrate a quantity-quality model as there are practitioners. Considering the wide range of approaches taken, it is surprising that calibration methods are not discussed more frequently in the literature. Few authors have described how they calibrated a model for a specific study and fewer still have stated explicity what criteria were used to determine when an adequate, or "best", fit was achieved. Descriptions of calibrations found in the literature follow.

Wanielista (14) used a single rainfall event to calibrate the U. S. Environmental Protection Agency's Storm Water Management Model (SWMM) for a 28 acre catchment in Orlando, Florida. Impervious area surface storage depth and resistance factor were adjusted to improve agreement between predicted and measure hydrographs. Agreement was very good (less than 0.2% error), probably because the catchment was about 96% impervious and the storm used was a relatively simple one. The area was discretized into 18 subcatchments and flows were routed through a system of 27 pipes, which further improved agreement. Very little was done in the way of quality calibration. Model default values were used for surface pollutant buildup rates. Predicted and measured mass emission rates were not compared.

The U. S. Army Corps of Engineers Hydrologic Engineering Center's Storage, Treatment, Overflow and Runoff Model for Metropolitan Masterplanning (STORM) was calibrated for Atlanta, Georgia, using storm event data gathered from four small, suburban catchments (5). Fifty-seven grab samples gathered from these four catchments during 15 storm events were analyzed. Pollutant accumulation rates found in STORM were adjusted to minimize the predictive error for the total lumped loads from all four catchments. According to Holbrook, the predictive error for each pollutant did not exceed 13% of the measured value. No information was given as to how the predictive error was calculated.

Young (16) suggested that a general method of calibration should involve adjustment of model parameters to produce dependent variable predictions that have minimum standard error of estimate with respect to known values of the dependent variables. Young did not indicate how large a data base would be required to accomplish a reliable calibration.

The User's Manual (6) for the U. S. Environmental Protection Agency Storm Water Management Model (SWMM) states that once the model is calibrated for one storm event, little adjustment is needed for others.

OBJECTIVES OF RESEARCH

One of the motivations for this research was a belief that the state-of-the-art of stormwater management modeling does not allow single storm calibration. Random storm variations and lack of knowledge concerning basin characteristic parameters require that several storms be used for calibration and, if possible, several more storms for verification. Only then can the model be used with confidence.

The objectives of this research are to show how a stormwater management model should be calibrated and verified for an urban basin using measured quantity and quality data from several storm events, and how the model can then be used to predict urban stormwater pollutant loads. Weaknesses of using single storm event calibration will be pointed out.

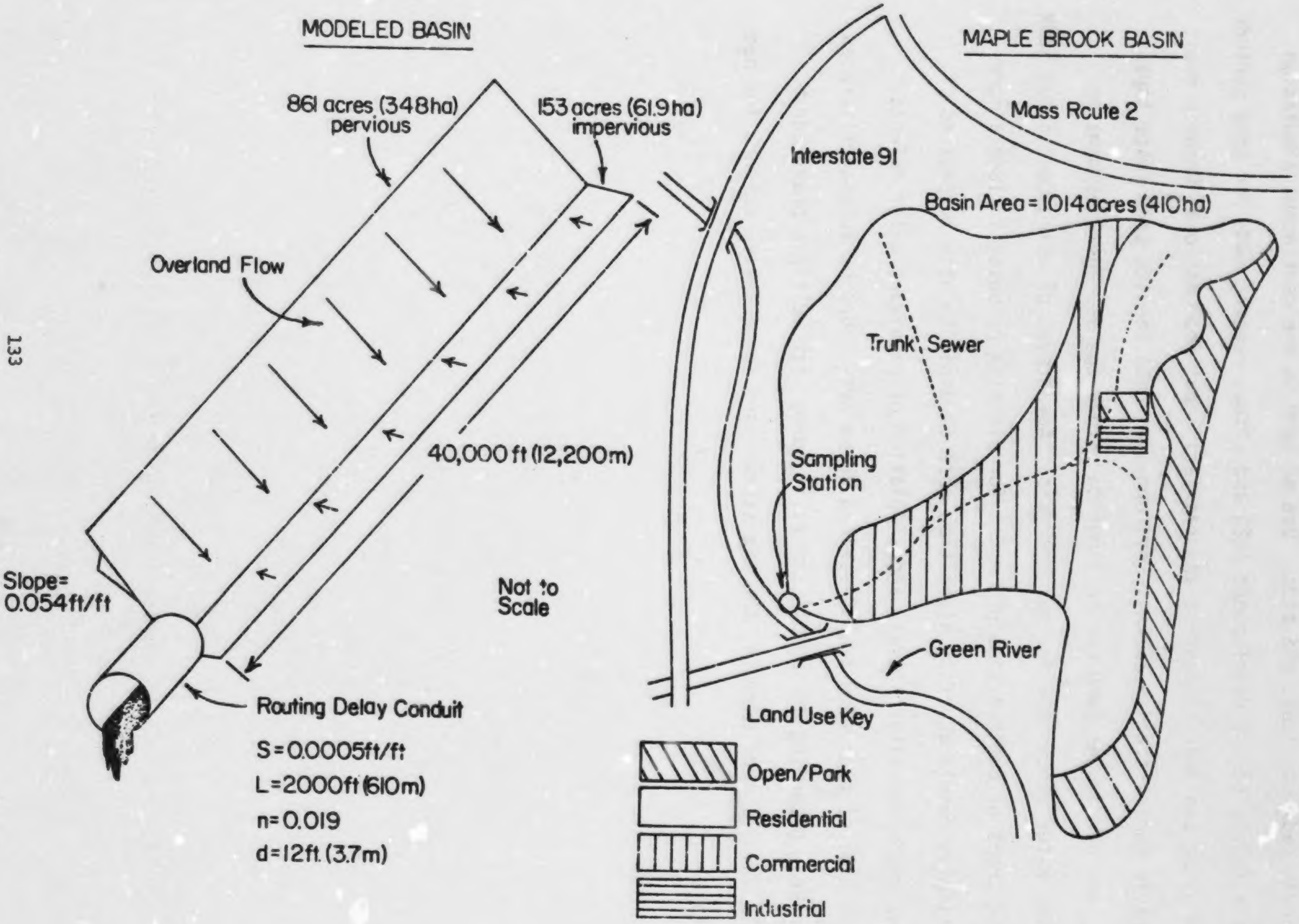
RATIONAL APPROACH TOWARD CALIBRATION

In order to generate discussion which will lead to more standardized methods for calibrating stormwater management models, a new approach toward calibration has been developed and applied to an urban basin. In developing this approach, a basic premise was that quantity and quality portions of models should be separated and calibrated independently. Following calibration, the model could be recombined and used in its normal manner.

Because each portion of the model is being calibrated separately, it is not necessary to use the same data set for both. This provides some flexibility as to what field data are acceptable for calibration purposes. The main concern is that each data set is representative of the average response of the basin.

The Storm Water Management Model (SWMM) was chosen to illustrate the approach. SWMM was chosen because of its availability to consultants and planners and because it is representative of the state-of-the-art in stormwater modeling. Quantity and quality subroutines of SWMM were separated and calibrated using storm event data taken from the 1014 acre (410 ha) Maple Brook basin (separate sewers) in Greenfield, Massachusetts (see Figure 1). Measured rainfall and sewer flow data were used to calibrate the quantity portion. The measured sewer flow data was used as input to the quality subroutine and the predicted pollutant mass emission rates were calibrated against measured mass emission rates. Model parameters were adjusted to produce mimimum error in total runoff and mass emissions and sums of peak flow rates across all calibration storms. Standard errors of estimate and normalized standard errors of estimates were tabulated for each run t indicate relative accuracy of calibration between runs. Independently calibrated subroutines were recombined and a preliminary verification accomplished using an independent set of data. The SWMM Runoff Block was then run in a continuous simulation mode to

Figure 1. -- Comparison of Maple Brook Basin with Modeled Basin.


MODELED BASIN
861 acres (348 ha) pervious
153 acres (61.9 ha) impervious
Overland Flow
40,000 ft (12,200 m)
Slope= 0.054 ft/ft
Not to Scale
Routing Delay Conduit
S = 0.0005 ft/ft
L = 2000 ft (610 m)
n = 0.019
d = 12 ft. (3.7 m)
MAPLE BROOK BASIN
Mass Route 2
Interstate 91
Basin Area = 1014 acres (410 ha)
Trunk Sewer
Sampling Station
Green River
Land Use Key
Open/Park
Residential
Commercial
Industrial

estimate yearly pollutant loadings from stormwater runoff in Greenfield, Massachusetts. Pollutants considered were suspended solids, BOD_5, total phosphorus, cadmium, lead, and zinc. Use of SWMM in the continuous simulation mode is a fairly recent development (12) and allows running SWMM for long periods of time using one hour time steps and rainfall input from National Oceanic and Atmospheric Administration (NOAA), Environmental Data Service card image tapes. Output from continuous SWMM can be input to either the Storage/Treatment or Receiving Water Blocks for further analysis. Advantages of using continuous SWMM over the Crops of Engineers' STORM model for planning purposes include a more sophisticated runoff quantity algorithm, more flexibility with respect to storage/treatment alternatives, and simulation of receiving water effects.

The sums of peak flowrates described above were found by summing flowrates at each peak occurring during all storms. Storms with multiple peaks added multiple values. For example, the six storms used for quantity calibration had 12 peaks.

GREENFIELD STUDY AREA

The Town of Greenfield is located in the northwest central portion of Massachusetts, at the confluence of the Green, Deerfield, and Connecticut Rivers. Greenfield has a total land area of 21 square miles (54 km^2) and a population of 18,500. Land use varies from open, undeveloped land to the concentrated central business district typical of most New England towns. Light industry is scattered throughout several portions of the town.

Approximately 80 percent of the urbanized area of Greenfield is drained by the Maple Brook separate storm sewer system. This sewer system evolved as the urban area developed. The main trunk sewer was once a streambed. Elements of the sewer system began appearing when culverts were installed where streets crossed the brook. These culverts, many of which were constructed before the turn of the century, were generally of rough stone rubble construction with a Manning roughness factor of approximately 0.025. Tributary storm drains were laid under streets perpendicular to the brook as the need arose. Eventually, as development increased, the brook sections between the cross streets were enclosed. In many cases these enclosed sections were made larger and smoother than the older culvert sections on either end. A schematic of the trunk sewer system is shown in Figure 1.

For this particular study, the entire Maple Brook basin was considered to be one subcatchment. Average slope, percent impervious area and gutter length were computed from data acquired for an earlier system analysis study which divided the basin into 76 subcatchments. Characteristic width was taken as twice the length of the main drainage channel through the basin, as was suggested in the SWMM User's Manual (6). No sewer routing was included in the initial configuration.

QUANTITY CALIBRATION

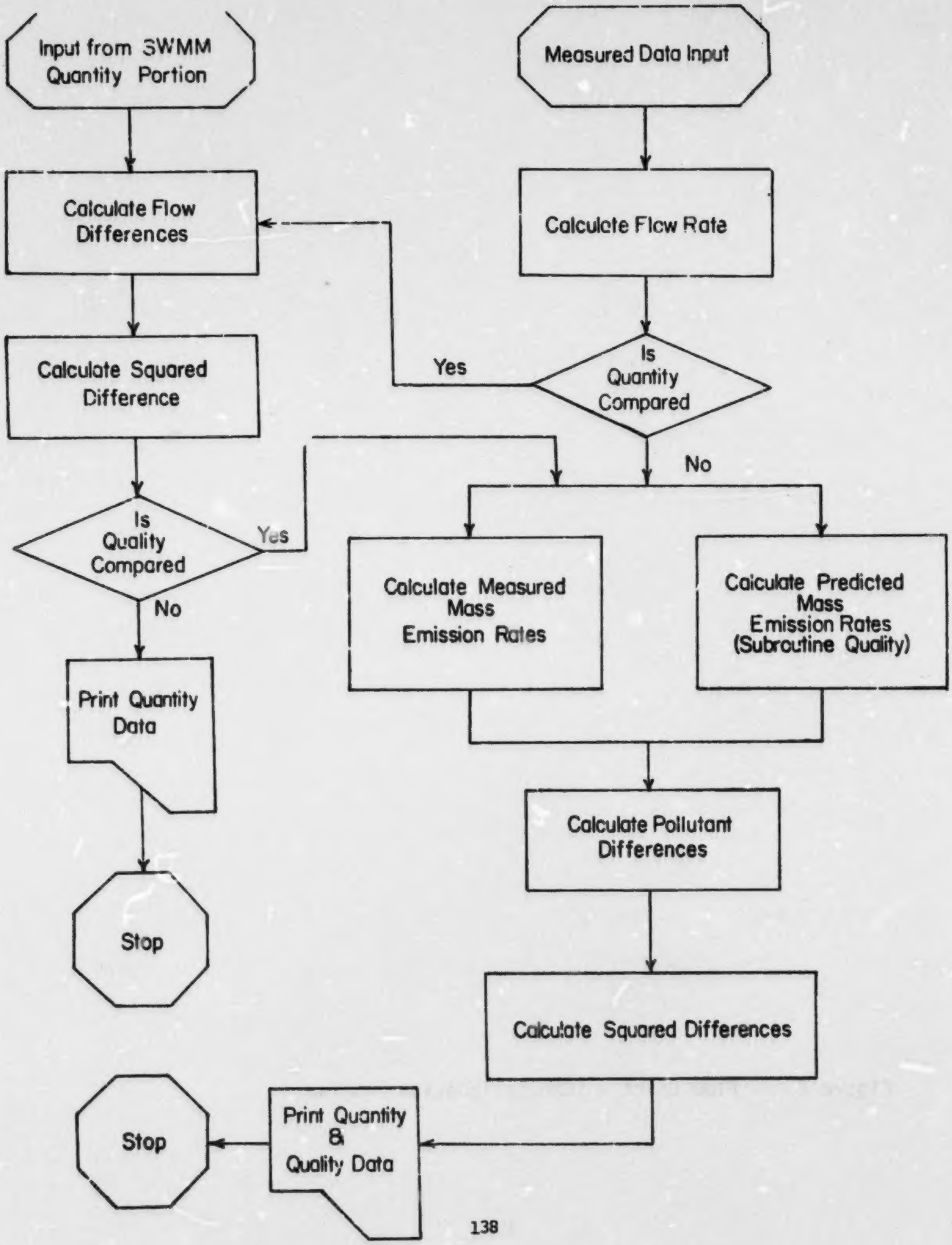
The objective of both the quantity and quality calibrations was to fit the model to average catchment conditions. During calibration it was not necessary to achieve close agreement of measured and predicted data for individual storm events. The emphasis was on results integrated over the entire calibration data set. If the assumption of representative data was valid, the calibrated model would give accurate predictions of average annual runoff and pollutant loadings.

Calibration criteria established were: less than one percent error between measured and predicted total volume of runoff and less than one percent error between the sums of measured and predicted peak flow rates. The first criterion was important because it measured long term volumetric discharge accuracy which was important in the prediction of receiving waters response. The second criterion indicated accuracy of short term volume predictions which would be important for estimating storage/treatment requirements.

To aid in calibration, an auxiliary program was written to compare measured and predicted flow and pollutant data and to compute representative statistics. A flow chart of this program is presented as Figure 2. This program could be used to compare quantity data alone, quality data alone, or both together. Subroutine QUAL, used to predict mass emission rates, contained the same logic found in subroutine QSHED1 of SWMM.

Several statistics for measuring the accuracy of fit between measured and predicted data were considered. Chow (2) has stated, "the goodness of fit of a function, fitted to the observed points by any procedure, can be measured and tested approximately by the χ^2 parameter." Young (16) has recommended that the standard error of estimate be used. In a personal communication, Koenig suggested using a function of the relative error, i.e. (predicted-measured)/measured.

Figure 2. -- Flow Chart - SWMM Calibration Program.


Input from SWMM Quantity Portion
Measured Data Input
Calculate Flow Differences
Calculate Flow Rate
Calculate Squared Difference
Yes
Is Quantity Compared
No
Is Quality Compared
Yes
No
Calculate Measured Mass Emission Rates
Calculate Predicted Mass Emission Rates (Subroutine Quality)
Print Quantity Data
Calculate Pollutant Differences
Stop
Calculate Squared Differences
Stop
Print Quantity & Quality Data

Although the standard error of estimate was developed for regression analysis, it was a frequently used statistic that appeared to have general applicability even though the model used was not statistical. Thus it was chosen as a representative statistic to measure the accuracy of fit between the measured and predicted data. The standard error of estimate had the form

$$SEE = \left[\frac{\sum_{i=1}^{n}(p_i-m_i)^2}{n-2}\right]^{1/2} \qquad (1)$$

in which

n = number of predicted and measured data points;

p_i = predicted value of ith data point; and

m_i = measured value of ith data point.

It expressed the degree of scatter in the measured data (7). A small SEE would indicate good agreement between the measured and predicted data, and thus an accurate calibration.

Flow data for calibration were gathered at the outfall of the Maple Brook sewer using a sharp crested, suppressed weir. Stage was recorded continuously by a Manning T-1000 transmitter coupled with a Rustrak strip chart recorder. Stage was converted into flow by a computer program that corrected the standard weir equation for velocity of approach. A Weathermeasure P501 rain gage was placed approximately in the center of the basin. Rainfall was recorded in 0.01 inch (0.025 cm) increments with a Rustrak recorder.

Data from six storm events were utilized for the quantity calibration. Table 1 gives the characteristics of the measured rainfall-runoff data. A wide range of rainfall intensities and durations were used so the calibration should be representative of conditions occurring on the catchment. Total rainfall of 3.94 inches (10.0 cm) represented approximately 10 percent of the average annual rainfall

TABLE 1. Measured Rainfall-Runoff Data

Storm Date (1)	Total Rainfall, in inches (2)	Duration, in minutes (3)	$\frac{\text{Volume}}{\text{Duration}}$, in inches per hour (4)	Total Runoff, in inches (5)	Runoff Coefficient, (6)
17 Oct 75	1.71	980	0.10	0.30	0.17
7 May 76	0.38	70	0.33	0.07	0.19
1 June 76	0.22	420	0.03	0.02	0.10
7 July 76	0.34	115	0.18	0.04	0.11
11 July 76	0.43	260	0.10	0.06	0.14
8 Oct 76	0.86	580	0.09	0.06	0.07
Verification Storm 17 Sept 76	0.36	260	0.08	0.05	0.15
Verification Storm 4 April 77	0.27	300	0.05	0.01	0.05

for Greenfield. The average runoff coefficient, weighted by storm event rainfall, was 0.14.

Initial catchment parameters for input to SWMM were derived from an earlier, detailed discretization of the Maple Brook basin used for sewer system analysis (7). Percent impervious area and slope were weighted according to area to arrive at representative values for the 1014 acre (410 ha) basin. The average slope remained constant throughout calibration at 0.054 ft/ft. Impervious area was initially calculated as 23.3 percent. Characteristic width was initially taken as twice the length of the main drainage channel through the basin, or 40,300 feet (12,290 m). Default values found in SWMM were used for resistance factors, depression storage depths and infiltration rates. If necessary, these parameters could be adjusted during calibration.

Prior to commencing calibration, it was necessary to ascertain model output sensitivity to changes in input parameters. Four studies have analyzed the sensitivity of SWMM (4,6,7,10). For each of these studies, all parameters except one were held constant. Then the parameter being tested was varied through a range about the accepted mean value. The ratios of parameter values and total predicted runoff volumes for the first three studies cited were calculated and are in Table 2. The Procter and Redfern study indicated similar degrees of sensitivity. Percent impervious area was the most sensitive parameter. Sensitivity of the other parameters was somewhat variable depending on the characteristics of the basin modeled and the range of parameter values used. With respect to total volume, the output was insensitive to pervious area depression storage and impervious area Manning's n, and was quite insensitive to characteristic width. The latter two parameters may, however, have had some effect on hydrograph

TABLE 2. Sensitivity Analysis Results

Parameter (1)	Graham et al.		Huber et al.		Jewell	
	Range (2)	Total Volume (3)	Range (4)	Total Volume (5)	Range (6)	Total Volume (7)
Percent Impervious Area	7:1	2.01:1	-	-	4:1	3.58:1
Pervious Area Minimum Infiltration Rate	1:10	1.82:1	1:150	1.75:1	-	-
Characteristic Width	1:79	2.00:1	-	-	2:1	1.03:1
Pervious Area Manning's n	1:4	1.36:1	1:150	1.00:1	-	-
Impervious Area Depression Storage	1:4	1.24:1	1:200	1.22:1	1:7	1.28:1
Pervious Area Depression Storage	1:4	1.02:1	1:50	1.00:1	-	-
Impervious Area Manning's n	1:3	1.02:1	1:100	1.02:1	1:2.3	1.02:1

shape. Impervious area depression storage may have a significant effect on total volume of runoff, especially for low rainfall volume storm events. Model output was sensitive to infiltration rates and pervious area Manning's n for some basins, however, analysis has shown that pervious areas of the Maple Brook basin did not contribute a significant amount to runoff. Thus the model output for the Greenfield basin was insensitive to any changes in pervious area parameters.

From these sensitivity analyses, it was deduced that percent impervious area would be the primary volume calibration factor. Characteristic width, impervious area Manning's n, or possibly a routing conduit could be used to change the shapes of the hydrographs and improve agreement between sums of measured and predicted peak flows.

No program has been written that will optimize parameter values within a multi-parameter model such as SWMM. If an optimization model could be written, it would be complex and expensive to run. Also, the data base available for calibration would probably not be sufficient to warrant using sophisticated optimization techniques. Thus model calibration remains a subjective process, aided by knowledge of model sensitivity and experience gained in previous calibrations. The accuracy of calibration possible depends on the degree of discretization used. A detailed discretization of the Maple Brook basin into 76 subcatchments and routing of the flows through the Transport Block would afford the opportunity of a more accurate calibration because of the more realistic portrayal of the basin. The calibrated model using a detailed discretization would yield better results for individual storm events but would be expensive to run for long periods of time. For planning purposes (long term simulation) it is much less important to calibrate for individual storm events so the whole basin can be adequately portrayed by one subcatchment.

Seven calibration runs were made before the criteria stated previously were met. Table 3 gives the parameters changed and the results for each run. Figure 3 shows the convergence of the volume and sum of peaks criteria and reduction in standard error of estimate. Figure 1 shows a schematic of the basin as modeled.

The uncalibrated model (run 1) predicted 60 percent more flow than was measured. Predicted peak flowrates were occurring before measured peak flowrates and the sum of the predicted peak flowrates was 110 percent high. To improve total volume agreement, the percent impervious area was reduced by a factor equal to the V_m/V_p ratio. This method of adjusting percent impervious area was carried through subsequent calibration runs. To improve temporal and magnitude agreement of peak flow rates, characteristic width and impervious area Manning's n were adjusted during the second and third runs, respectively. It became clear during subsequent runs that reasonable adjustment of these parameters would not produce agreement between sums of measured and predicted peak flow rates. Therefore during the fourth run, a single, 2000 foot (610 m) conduit with a slope of 0.0005 ft/ft was added to provide routing delay. Size and Manning's n of the conduit were adjusted until the criterion of ± one percent between sums of measured and predicted peak flows was met. The final ratio of predicted and measured volumes (1.000) and the final ratio of sums of measured and predicted peak flow rates (1.007) were both within the ± one percent criteria. Note that the standard error of estimate had a minimum value after run five then increased slightly through run seven. Although the SEE is an indicator of accuracy of fit, it is not necessarily minimized by meeting the calibration criteria set. This is consistent with the objective of calibrating for long term agreement rather that individual storm event reproduction. Although both criteria have been met, there is still some time lag between predicted and measured peaks. If these time lags could be eliminated, it would improve the SEE

TABLE 3. Quantity Calibration Data

		Run						
(1)	(2)	1 (3)	2 (4)	3 (5)	4 (6)	5 (7)	6 (8)	7 (9)
Subcatchment Parameters	Percent Impervious Area	23.3	14.0*	14.6*	14.5*	14.8*	15.1*	15.1
	Characteristic Width, in feet	40300	30000*	30000	30000	40000*	40000	40000
	Impervious Area Manning's n	0.013	0.013	0.03*	0.03	0.03	0.013*	0.013
Conduit Parameters	Diameter, in feet	-	-	-	8.0	12.0*	12.0	12.0
	Manning's n	-	-	-	0.013	0.025*	0.025	0.019*
Criteria	V_m/V_p	0.626	1.044	0.992	1.023	1.017	1.007	1.000
	$\Sigma P_m/\Sigma P_p$	0.477	0.774	0.852	1.066	1.165	1.080	1.007
	Standard Error of Estimate**	21.00	13.50	11.43	8.05	7.56	7.88	8.40

*Indicates parameter has been adjusted from previous run. **Units of cubic feet per second.
Note: 1 ft = 0.305 m; 1 cfs = 0.028 m^3/m.

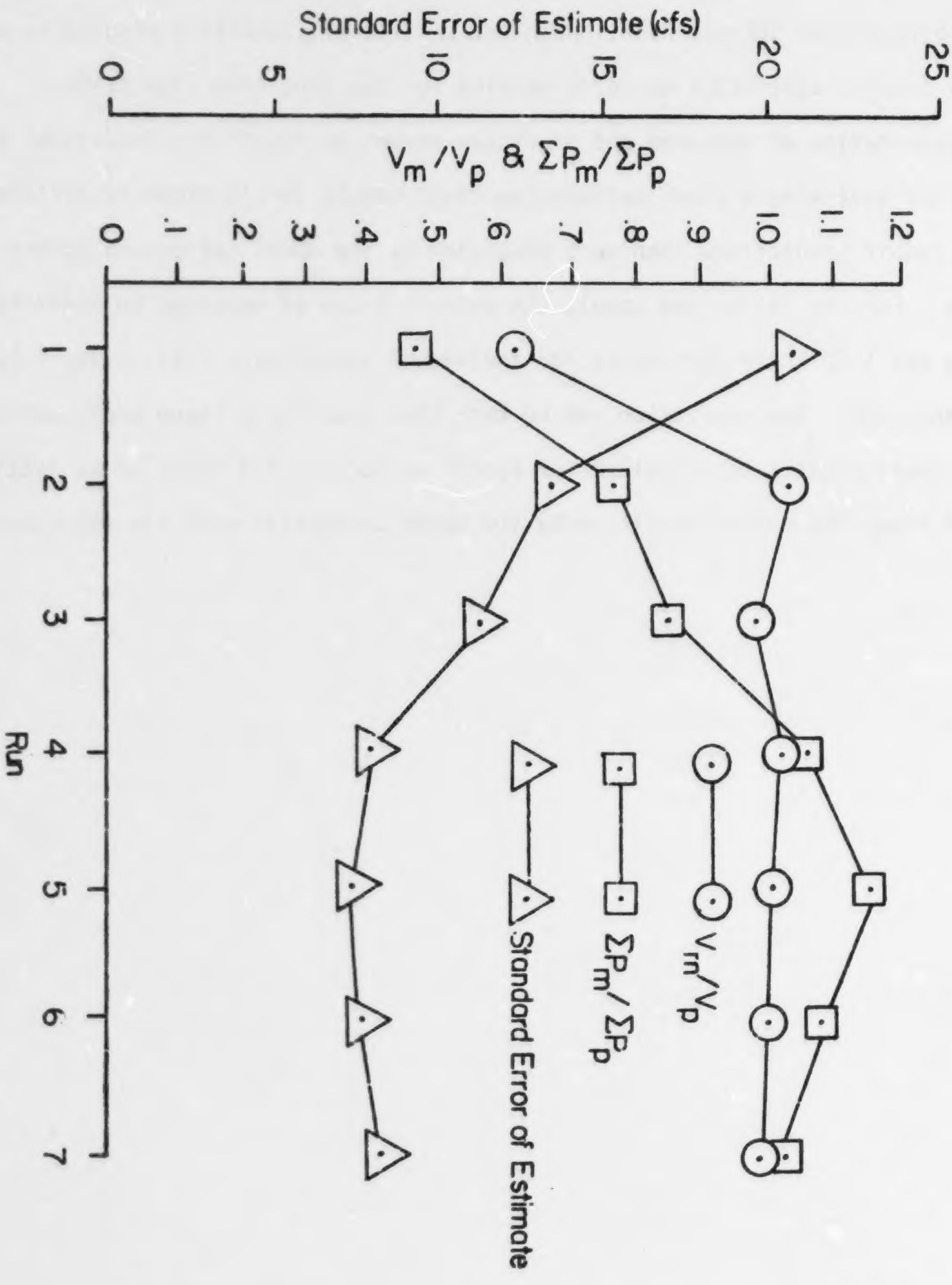


Figure 3. -- Calibration Criteria and Statistics.

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considerably. To demonstrate, all predicted flow rates were lagged by 5 minutes and the resulting SEE was 7.87. However, artificially shifting results to achieve better looking statistics would do nothing for the long term simulation.

Examination of measured and predicted volume of runoff for individual storms indicated that single storm calibration could result in signficantly different total runoff predictions than were generated by the model calibrated across six storms. For the calibrated model, the overall ratio of measured to predicted volume was 1.00 while the ratios for individual storms were 1.43, 0.75, 1.20, 0.53, 0.71 and 1.01. The conclusion can be made that the single storm event calibrated model could predict total volumes of runoff as much as 1.4 times or as little as 0.5 times the volume predicted by the model calibrated with six storm events.

QUALITY CALIBRATION

As described earlier, the calibration program used in this investigation utilized an adaptation of SWMM subroutine QSHED1, driven by measured flow data, to simulate pollutant washoff from the watershed. Measured pollutant concentrations and simulated data were converted to mass emission rates (lbs/min) for comparison. The calibration program was structured so that comparisons could be made at time intervals corresponding to the sampling times of measured data.

Maintenance of the ratio of predicted to measured mass emissions (P/M) at 1.0 was the primary quality calibration criterion. All storms were weighted equally when predicted mass emissions were adjusted. The normalized standard error of estimate was utilized for goodness of fit comparisons between different pollutants.

Samples for quality analysis were collected using a Manning S-4000 automatic sampler in combination with a Manning T-1000 dipper-transmitter stage recorder. The sampler intake line was located at approximately middepth, 20 feet (6.1 m) upstream of a 3.0 foot (.9 m) sharp crested weir. In automatic mode, the sampler was activated at a preselected stage height and took two 500 ml samples at 15 or 30 minute intervals, until 24 samples had been drawn. Manual mode sample collection was also utilized, allowing the flexibility of choosing varying time intervals depending on hydrograph dynamics.

Parameters determined for the quality analysis were suspended solids (SS), biochemical oxygen demand (BOD_5), total phosphorus, and three dissolved metals; zinc, cadmium, and lead. Biochemical oxygen demand[1] and suspended solids were chosen because they are common indicators of the impact of urban runoff on receiving waters. Total phosphorus can also be an important measure of urban runoff pollution, particularly if the basin drains into a slow moving stream or lake where algal blooms or eutrophication may be problems. The choice of dissolved metals to be monitored was based on a tradeoff between mass loadings in urban runoff and toxicity to mankind. The importance of a data base for both heavy metals and nutrients is pointed out in a recent criticism of the EPA 1976 "Needs Survey" for treatment of combined and stormwater flows which indicated that a lack of consideration for these constituents would be an error (15).

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[1] Analysis of several samples indicated that BOD_5 was not influenced by toxic materials for the Maple Brook basin.

Samples for suspended solids, BOD_5, and total phosphorus determinations were preserved in an ice bath until analyses were performed. Samples for metals analyses were preserved with 1 part per 100 of 50 percent nitric acid solution. All analyses were accomplished using approved methods form Standard Methods (13). The analyses performed were: suspended solids, non-filterable total residue; BOD_5, five day incubation at 20°C and Winkler titration method for DO determination; total phosphorus, ascorbic acid spectrophotometric method; zinc, direct flame spectroscopy (Perkin-Elmer model 303); lead, three replicates through graphite furnace and model 303 sensing unit (Perkin-Elmer HGA-70 graphite furnace); and cadmium, same as lead.

Pollutant concentrations were multiplied by measured flow rates to determine instantaneous mass emission rates for comparison during calibration. Measured flow rates were also used to drive the SWMM surface pollutant washoff function (Equation 4). Rainfall data were not necessary for quality calibration. The quality data for this study was collected from May 1976 to December 1976. During this period, reliable data was obtained for five storm events; on the dates of 6/1/76, 7/11/76, 9/17/76, 10/20/76, and 11/28/76.

For the initial uncalibrated SWMM simulation run, data was prepared according to procedures outlined in the SWMM User's Manual, Version II (Huber, et al. 1975). Surface quality data required included gutter length, street sweeping practices, catchbasin characteristics, and number of dry days antecedent to the storm event.

For purposes of this investigation, the basin was modeled as a single 1014 acre (410 ha) subcatchment. The total gutter length for the basin was found to be 141,700 feet (43,220 m). There were relatively few catchbasins in the Maple Brook basin, so their effects were not modeled. Street sweeping frequency was approximately 60 days with one pass per cleaning.

SWMM predicts pollutant buildup as a linear function:

$$P_j = DDFACT*QFACT_j*ADD \tag{2}$$

in which

P_j = mass of pollutant j on basin at start of storm;

DDFACT = dust and dirt accumulation rate for particular land use (lb/day/100 ft of curb);

$QFACT_j$ = fraction of dust and dirt that is pollutant j; and

ADD = antecedent dry days (defined in SWMM as number of antecedent days for which cumulative rainfall is less than 1.0 inch).

Pollutant buildup is corrected for periodic sweeping, when the antecendent dry period exceeds the street sweeping frequency.

The definition of ADD presented in Equation 3 is consistent with the assumption made to derive the pollutant washoff decay coefficient of Equation 4. The assumption was made that 0.5 inches (1.3 cm) would wash off 90 percent of the pollutants, therefore 1.0 inch (2.5 cm) would wash off 99 percent. Since nearly all rainfall runs off impervious areas and pollutant washoff is only modeled for impervious areas, the 1.0 inch (2.5 cm) cumulative rainfall criterion follows. While initially there was no reason to doubt the ADD definition or the linear buildup rate, subsequent calibration efforts indicated that there was some fixed antecedent period beyond which little additional pollutant accumulation would take place.

Default values for DDFACT and QFACT for various land uses are contained in the model (6). These values were derived from an American Public Works Association study of Chicago (1). The SWMM user has the option of inputing his own values for DDFACT and QFACT based on local data.

Default values of DDFACT and QFACT were weighted according to area of the basin in a particular land use to arrive at initial values for calibration. Area weighted DDFACT for this basin was 1.39 lbs of dust and dirt/day/100 ft of curb (2.06 kg/day/100 m). Area weighted QFACT for BOD_5 was 5.29×10^{-3}. Suspended solids QFACT was assumed to be 1.0, as was done in SWMM. QFACTs for total phosphate were weighted to give an initial estimate for total phosphorus loading. Since no QFACTs were available for dissolved metals, these were estimated based on the concentration ranges of the measured data. All initial QFACT*DDFACT values used are given in Table 4.

Data from five storm events were used to calibrate all pollutants except suspended solids. Suspended solids data for the storm of 9/17/76 were saved for preliminary verification, and suspended solids data from the storm of 11/28/76 were not used for calibration because they were believed to be atypical.

The results of the initial simulation run are presented in Table 4. The P/M ratios for suspended solids and BOD_5 of 0.259 and 0.125 respectively indicate that using the uncalibrated model for Greenfield would cause large errors in total mass emissions. Predicted mass emissions for total phosphorus were two orders of magnitude lower than measured loadings (P/M ratio = 0.0249); probably because the SWMM PO_4 loadings were used as a first estimate of total phosphorus loadings. Initial run metals emissions cannot be compared because the initial loadings were not derived from existing model loadings.

TABLE 4. Quality Calibration: Initial and First Runs

(1)	(2)	SS (3)	BOD_5 (4)	P (5)	Cd (6)	Pb (7)	Zn (8)
Initial Uncalibrated Run	QFACT*DDFACT, in lb/day/100 ft of Curb	1.39	7.35×10^{-3}	6.95×10^{-5}	1.39×10^{-5}	3.48×10^{-5}	4.87×10^{-5}
	Predicted/Measured Mass Emissions	0.259	0.125	0.0249	2.305	0.161	0.114
	Normalized Standard Error of Estimate	6.42	1.57	2.47	3.50	3.10	2.05
First Calibration Run	QFACT*DDFACT, in lb/day/100 ft of Curb	5.36	5.91×10^{-2}	2.68×10^{-3}	6.03×10^{-6}	2.16×10^{-4}	4.27×10^{-4}
	Predicted/Measured Mass Emissions	1.00	1.00	1.00	1.00	1.00	1.00
	Normalized Standard Error of Estimate	2.80	1.11	1.47	3.78	2.79	1.47

Note: 1 lb/day/100 ft of curb = 1.485 kg/day/100 m of curb.

After completion of the initial uncalibrated run, the next step was the adjustment of the model parameters to satisfy the total mass emission calibration criteria. Adjustment of QFACT for each pollutant proved to be the easiest way to make predicted and measured mass emissions agree. Although this produced some unrealistic QFACT values (QFACT for suspended solids became greater than 1.00) the only result of interest was the product of QFACT and DDFACT (lb of pollutant/day/100 ft of curb), so QFACT could assume any convenient value.

The method of adjustment used was to divide the old QFACT by P/M to arrive at a corrected QFACT. In effect, all mass emission rates were increased by the factor 1/(P/M). When summed across all calibration storms, the adjusted mass emissions agreed with measured emissions, as is shown in Table 4. This initial adjustment improved the normalized standard error of estimate of all pollutants except cadmium. Cadmium may not follow an exponential decay model and other models should be investigated.

After satisfying the mass emissions criteria, further calibration efforts focused on achieving a better curve fit. Results are shown in Table 5. To permit comparison of calibration accuracy between pollutants, the SEE for each pollutant was divided by the average measured mass emission rate for that pollutant. Thus the SEE became nondimensional and was called the normalized standard error of estimate.

The first calibration run seemed to indicate a better fit for BOD_5, total phosphorus, lead and zinc than for suspended solids. A review of the development of the SWMM model revealed that during initial testing it was necessary to add an availability factor to account for the portion of dust and dirt that is not normally available for the production of suspended solids. Thus the equation for prediction of suspended solids washoff becomes:

TABLE 5. Further Attempts at Quality Calibration

(1)	(2)	$SS_{(3)}$	BOD_5 (4)	P (5)	Cd (6)	Pb (7)	Zn (8)
First Calibration Run (Used for Continuous SWMM)	Mass Loading, in lb/day/100 ft of Curb	5.36	5.91×10^{-2}	2.68×10^{-3}	6.03×10^{-6}	2.16×10^{-4}	4.27×10^{-4}
	Normalized SEE	2.80	1.11	1.47	3.78	2.79	1.47
No Availability Factor	Mass Loading	0.479	5.91×10^{-2}	2.68×10^{-3}	6.03×10^{-6}	2.16×10^{-4}	4.27×10^{-4}
	Normalized SEE	3.65	1.11	1.47	3.78	2.79	1.47
Antecedent Dry Days Criterion 0.5 inches(1.3 cm) Cumulative Rainfall	Mass Loading	5.36	7.62×10^{-2}	3.36×10^{-3}	6.81×10^{-6}	2.68×10^{-4}	5.65×10^{-4}
	Normalized SEE	2.80	0.96	1.34	3.86	2.75	1.34
Constant Pollutant Loading ADD=10 days	Mass Loading	4.48	0.111	5.30×10^{-3}	1.25×10^{-5}	5.45×10^{-4}	9.61×10^{-4}
	Normalized SEE	2.86	0.765	0.971	2.40	2.45	1.10

Note: 1 lb/day/100 ft of curb = 1.485 kg/day/100 m of curb.

$$SS_m = \underbrace{M_m(0.057 + 1.4\ R_m{}^{1.1})}_{\text{Availability Factor}}\ \underbrace{[1-\exp(-k\ R_m\ \Delta t)]}_{\text{Exponential Decay}} \qquad (3)$$

in which

SS_m = pounds of suspended solids washed off during time step m;

M_m = pounds of suspended solids on subcatchment at start of time stem m;

R_m = runoff rate (inches/hour);

k = washoff decay coefficient (inches^{-1}); and

Δt = simulation time step size (hours).

NOTE: Calculation of washoff for other pollutants is similar but without availability factor.

The availability factor was a function of runoff rate and was developed using data from Cincinnati, Ohio (9). Since the availability factor was derived from site specific data, its generality was questionable. Therefore, it was removed from the model and suspended solids mass loadings were readjusted to maintain agreement with measured data. The calculated surface mass loading was lower but the normalized standard error of estimate was higher, indicating that the availability factor improved suspended solids predictions for the Greenfield data. Generality of the availability factor is still in question and an indepth study would be useful.

The next attempt to improve mass emission curve fit involved adjustment of the washoff decay coefficient (k) in equation 4 from its default value of 4.6. Changing k changed the shape of the mass emission curve, however, when total predicted mass emissions were readjusted to agree with measured data, little improvement was made in the standard error of estimate over a wide range of k values (k = 0.1 to k = 20). Indications were that SWMM was insensitive to changes in value of k over the range of runoff rates normally encountered. Further research is needed, however, for the present time, it can be concluded that the washoff decay coefficient is not an efficient calibration variable.

The effects of changing the antecedent dry days definition were also investigated. Changing the criterion for cumulative rainfall from 1.0 inch (2.5 cm) to 0.5 inch (1.3 cm) improved the standard error of estimate for all pollutants except cadmium by the amounts shown in Table 5. Next, the cumulative rainfall criterion was completely removed and a constant mass of pollutants was made available at the start of each storm. To accomplish this, the ADD for each storm was set equal to 10 days. For constant ADD, standard errors of estimate decreased for all pollutants except suspended solids. Suspended solids data for the storm of 11/28/76 had not been included in the initial analysis because of the long dry period prior to that storm (34 days). These data were reintroduced into the analysis when a constant antecedent dry period was used. Results indicated that the pollutant buildup rate was not linear and that there was a limit to the mass of pollutants that could accumulate between storms, regardless of the length of dry period. Sartor and Boyd (11) came to the same conclusions when they examined pollutant buildup data from six different cities.

The data base for Greenfield was not sufficient to determine the functional form of the pollutant buildup equation or the maximum pollutant buildup possible. Therefore, the published definition of ADD was used and the results of the first calibration run were used for the calibrated model.

An examination of the individual storms used for quality calibration again indicated the weakness of single storm event calibration. For example, the BOD_5 measured to predicted mass emission ratios for the five storms used for calibration were 1.31, 1.63, 0.75, 2.67, and 0.52. Thus, if the quality portion of the model had been calibrated using only one storm, the annual BOD_5 loadings predicted would have been anywhere from 0.52 to 2.67 times those predicted by the model as calibrated over five storm events.

VERIFICATION

Data from storms occurring on 17 September 1976 and 23 April 1977 were used to verify flow and suspended solids predictions. Data from the 23 April 1977 storm was also used to verify BOD_5 predictions. Rainfall and runoff characteristics for these storms are shown at the bottom of Table 1.

Results of the verification were encouraging. The ratio of predicted to measured total runoff was 0.92 while the same ratio for the sum of peak runoff rates was 1.06. The ratios for total mass emission of suspended solids and BOD_5 were 0.80 and 0.72 respectively.

It would have been better to accomplish the verification using data from four or five storms of varying magnitude. However, the field data gathering program in Greenfield has been terminated and further verification data will not be available.

Data from a third storm was not used for verification because the short antecedent dry period (2 days) produced pollutant mass emission predictions that were much lower than measured mass emissions. The predicted to measured mass emission ratios for suspended solids and BOD_5 were 0.10 and 0.21 respectively. This provided further evidence that the pollutant accumulation rate was not linear. A greater mass of pollutant was available on the basin soon after a storm event than was predicted by the linear accumulation function. A need exists to investigate improved methods for predicting pollutant accumulation.

APPLICATION OF CALIBRATED MODEL

The calibrated model was used to predict annual stormwater pollutant loadings from the 1014 acre (410 ha) Maple Brook basin. Ten years of hourly rainfall record for Amherst, Massachusetts [14 miles (22.5 km) from Greenfield] were used to drive runoff calculations. No rainfall data tape was available for Greenfield but a check of annual precipitation indicated that the average rainfall for both locations was equal.

Annual loadings, average loadings, and standard deviations for the ten years simulated are given in Table 6. Suspended solids loadings generally followed annual precipitation, while loadings for the other pollutants showed little variation with annual rainfall. Lack of variability in BOD_5, phosphorus, cadmium, lead, and zinc indicated that the amount of runoff generated in Greenfield was sufficient to wash the basin clean each year. The variability of the suspended solids loading indicated that the converse was probably true for that pollutant. Some of the suspended solids built up on the basin each year were not washed off and became part of the basin.

Annual loadings of BOD_5 and phosphorus agree quite well with annual loading estimated from river sampling during an earlier study. DiGiano, et al. (3) sampled flows during three discrete storm events. In terms of a 365 day year, DiGiano estimated BOD_5 and phosphorus loadings of 30 and 0.8 $lb/mi^2/day$ (5.2 and 0.14 $kg/km^2/day$), respectively. Continuous simulation predicted a BOD_5 loading of 45 $lb/mi^2/day$ (7.9 $kg/km^2/day$) and a total phosphorus loading of 2.0 $lb/mi^2/day$ (0.35 $kg/km^2/day$). Comparison of the annual suspended solids and BOD_5 loadings from the Maple Brook basin with those coming from the Greenfield secondary sewage treatment plant indicated that stormwater contributed seven times as much suspended solids and one-fifth as much BOD_5 to the receiving waters as did the sewage treatment plant effluent.

TABLE 6. Yearly Loading Data

Year (1)	Rainfall, in inches (2)	Runoff, in inches (3)	C* (4)	SS, in pounds (5)	BOD_5, in pounds (6)	P, in pounds (7)	Cd, in pounds (8)	Pb, in pounds (9)	Zn, in pounds (10)
1950	42.07	5.99	0.14	$.146 \times 10^7$	$.258 \times 10^5$	$.117 \times 10^4$	2.63	94.5	186
1951	46.12	6.61	0.14	$.161 \times 10^7$	$.278 \times 10^5$	$.126 \times 10^4$	2.83	102	201
1952	40.02	5.78	0.14	$.139 \times 10^7$	$.261 \times 10^5$	$.119 \times 10^4$	2.66	95.6	189
1953	49.68	7.19	0.15	$.143 \times 10^7$	$.256 \times 10^5$	$.116 \times 10^4$	2.62	93.9	185
1954	43.59	6.20	0.14	$.130 \times 10^7$	$.257 \times 10^5$	$.117 \times 10^4$	2.62	94.1	186
1955	55.31	8.23	0.15	$.171 \times 10^7$	$.241 \times 10^5$	$.109 \times 10^4$	2.46	88.1	174
1956	40.24	5.74	0.14	$.138 \times 10^7$	$.263 \times 10^5$	$.119 \times 10^4$	2.68	96.3	190
1957	34.07	4.86	0.14	$.856 \times 10^6$	$.249 \times 10^5$	$.113 \times 10^4$	2.54	91.1	179
1958	41.43	5.90	0.14	$.132 \times 10^7$	$.244 \times 10^5$	$.111 \times 10^4$	2.49	89.2	176
1960	47.64	6.85	0.14	$.144 \times 10^7$	$.259 \times 10^5$	$.118 \times 10^4$	2.64	94.8	187
AVG	44.02	6.34	0.14	$.139 \times 10^7$	$.257 \times 10^5$	$.117 \times 10^4$	2.62	94.0	185
SS**	5.94	0.93	0.00	$.226 \times 10^6$	$.105 \times 10^4$	$.477 \times 10^2$	0.10	3.9	8

*C = Runoff coefficient

**SS = Standard deviation

Note: 1 in = 2.54 cm; 1 lb = 0.453 kg

From the results shown in Table 6, it appears that two or three years of continuous simulation would be sufficient to estimate yearly pollutant loadings for all pollutants with the possible exception of suspended solids. The extra cost of additional years of simulation is probably not warranted in light of the consistency of results.

Another useful application of the calibrated model would be to test alternative outfall control measures or receiving waters effects. Alternatively, loading factors could be converted back to individual land uses and the detailed discretization of the basin could be utilized to test alternative land use distributions and stormwater control measures within the basin. Use of the detailed discretization would require some further calibration of the quantity portion of the model but would require no further calibration of the quality portion.

SUMMARY AND CONCLUSIONS

A method has been described for calibrating coupled quantity-quality urban stormwater management models. By separating the quantity and quality portions of the model it is possible to calibrate each using only measured data. The calibrated model will be an accurate and economic tool for predicting alternative futures.

The simplicity of the calibration procedure used should be emphasized. Good quantity calibration can be achieved by adjustment of a few basin parameters and quality calibration involves only the adjustment of pollutant buildup rates for most cases. An advantage of the method used is that quantity and quality portions of the model do not have to be calibrated with the same data set.

It is hard to quantitatively compare this calibration method with other previously used calibration methods because of the lack of discussion of multi-storm calibrations in the literature and the lack of year long runoff quantity and quality data sets to check final outputs. If data for other multi-storm calibrations were available, the normalized standard error of estimate could be utilized to compare the relative accuracy of methods.

The Storm Water Management Model was designed to be a deterministic model. Theoretically it could be calibrated using any storm event with similar results (6). However, the variation between storms of measured to predicted flow volume and pollutant mass emission ratios (for the calibrated model) indicated that the model did not precisely portray actual basin conditions. Therefore, different storm events will result in different calibrations and different predictions. Calibrating for average conditions across several storms will reduce predictive error and increase confidence in results.

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APPENDIX II. - NOTATION

The following symbols are used in this paper:

ADD = antecedent dry days;

DDFACT = dust and dirt accumulation rate for particular land use, in pounds per day per 100 feet of curb;

k = washoff decay coefficient, in inches^{-1};

M_m = mass of suspended solids on subcatchment at start of time step m, in pounds;

m_i = measured value of ith data point, in pounds per minute;

n = number of predicted and measured data points;

P_j = mass of pollutant j on basin at start of storm, in pounds,

p_i = predicted value of ith data point, in pounds per minute;

$QFACT_j$ = fraction of dust and dirt that is pollutant j;

R_m = runoff rate during time step m, in inches per hour;

SEE = standard error of estimate;

SS_m = mass of suspended solids washed off during time step m, in pounds; and

Δt = simulation time step, in hours.

Subscripts

i = data point;

j = pollutant;

m = time step.

SUMMARY

Methodology for Calibrating Stormwater Models, by Thomas K. Jewell, Thomas J. Nunno, and Donald Dean Adrian. (EE). A method for calibrating coupled quantity-quality stormwater management models is presented. Quantity and quality subroutines are separated and each is calibrated using measured data. Model parameters are adjusted based on sensitivity analyses, until predicted and measured outputs are within stated calibration criteria.

KEY WORDS

Environmental engineering, Hydrology, Models, Regional planning, Sewers, Stormwater management, Storms, Urban planning, Water pollution

ABSTRACT

A method for calibrating coupled quantity-quality stormwater management models is presented. The U. S. Environmental Protection Agency Storm Water Management Model is used as an example. Quantity and quality subroutines are separated and each is calibrated using measured data. Data from six storm events are used for quantity calibration and data from five storm events are used for quality calibration. Model parameters are adjusted, based on sensitivity analyses, until predicted and measured outputs are within stated calibration criteria. The calibrated subroutines are then recombined and the coupled model is used to predict annual pollutant loadings. Loading rates are compared with loading rates predicted using a single storm event calibrated model.

A SIMPLIFIED CONTINUOUS RECEIVING WATER QUALITY MODEL

by

Miguel A. Medina Jr.
Department of Civil Engineering
Duke University
Durham, North Carolina 27706

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Users Group Meeting
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Ontario, Canada

INTRODUCTION

PROBLEM DEFINITION AND MANAGEMENT TOOLS

In a 1.67 square mile (433 hectare) urban watershed in Durham, North Carolina, it was found that the dissolved oxygen content of the receiving watercourse was independent of the degree of municipal waste treatment beyond secondary during storm flows (Colston, 1974). Approximately *one-half* of the stream miles in the United States are water quality limited and 30 percent of these stream segments are considered polluted to a certain degree with urban stormwater runoff (Field, *et al*., 1977). The implication is that, generally, secondary treatment of dry-weather wastewater flows is insufficient to meet desired receiving water quality standards: therefore, control of runoff pollution must be considered in areawide wastewater management plans and abatement programs. The results of a nationwide assessment of costs and related water quality impacts derived from non-point sources (Heaney, *et al*., 1977) were, among others, that:

- wet-weather flows represent at least 50 percent of the total wastewater flow from urban areas;
- a generalized optimization model, assuming linear costs, predicted primary type facilities are preferable only up to a 10 percent level of BOD removal for wet-weather flows, with a secondary type facility preferable for higher levels of control; and
- on a national average basis using BOD removal as the effectiveness parameter, approximately 39 percent of the combined sewer problem and 10 percent of the other wet-weather flows should be controlled before initiating tertiary treatment of point sources.

The study also confirmed that gross inadequacies exist in our present data base and conclusions are highly sensitive to simplifying assumptions necessary for successful simulation of complex physical processes occurring throughout our watersheds. Nevertheless, mathematical models are needed to predict variable responses to stochastic hydrologic phenomena.

The 208 planning effort (Section 208, PL 92-500) has established the need for various levels of urban water management analysis (Field, et al., 1977) to permit preliminary screening of municipal treatment alternatives. Four distinct levels of evaluation techniques, ranging from simple to complex procedures which can be integrated with one another, are summarized in Table 1. The first three levels essentially represent various degrees of planning detail with models running on hourly time steps for long simulation periods (years). Mathematical complexity and data requirements are kept at a minimum. Of course, detailed (single-event) SWMM is typically used with short time steps (minutes) and short simulation times (hours). Its data requirements are usually very substantial. The approach guiding the development of Level-III Receiving was that the cost-effectiveness of various treatment alternatives can be determined realistically only by a continuous analysis of the frequency of violation of water quality standards.

MODEL OVERVIEW

The essence of a rational water quality and quantity management program is the decision making process. The high cost of pollution control facilities, in terms of both energy utilization and financial burden, obligates the planning agency to select the optimal strategy for areawide wastewater management. Such a process must focus on a systematic procedure that identifies and defines: 1) the cause/effect relationships of the physical environment; 2) the economic realities of control alternatives; and 3) the benefits to be derived from implementation of these controls. A preliminary analysis that provides an approximation of system responses to proposed treatment measures should aid the selection of the best strategy for restoration of water bodies to accepted water quality standards. Such an analysis must never be interpreted as other than a guide to be tempered by professional judgment. The mathematical models applied need not incorporate all phenomena but rather should be relevant to the problem under consideration. The problem of specific interest is to assess the separate and combined effects of the major urban sources of water pollution upon the quality of the receiving waters. Oxygen concentration is considered the key to the quality of natural water bodies, although it certainly is not the only water quality indicator. Thus, the relative impact of these wastewater sources is appraised

Table 1. Levels of Urban Water Management Analysis
(modified after Field, et al., 1977)

Level I: a desktop calculator, statistical analysis procedure, no electronic digital computer required.

- University of Florida Methodology - permits the user to estimate the quantity and quality of urban runoff in the combined, storm and unsewered portions of each urban area
- Hydroscience, Inc. Methodology - use of a stormwater simulator and an analytical method based on probability distribution functions and statistical properties of rainfall, runoff, treatment and receiving water impact.

Level II: a simplified continuous simulation model for planning and preliminary sizing of facilities, developed by Metcalf & Eddy, Inc.

Level III: a refined continuous simulation model approach. Continuous hydrologic simulation models (e.g., STORM or continuous SWMM) which generate urban runoff hydrographs and pollutographs are followed by continuous receiving water impact analyses (Level III - Receiving model).

- Continuous SWMM - University of Florida
- STORM - for Corps of Engineers by Water Resources Engineers, Inc.
- Level III - Receiving - Duke University

Level IV: a sophisticated single event simulation model, EPA SWMM developed by Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc.

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by their effect on the dissolved oxygen concentrations downstream from the urban area. It is of further interest to distinguish clearly between the two types of urban stormwater runoff, separate sewer flow and combined sewer overflow, and their relative pollutional impacts. In essence, the mathematical model must be responsive to the land use, hydrology, and climatology of the drainage area while performing the following functions:

- generate stormwater runoff pollutant loads and dry-weather sanitary flow pollutant loads;
- simulate the pollutant removal efficiency of various treatment schemes;
- simulate the conveyance system, including mixing in combined sewers of wet- and dry-weather pollutants;
- mix the various pollutant inflow combinations with pollutants already in the receiving water (from upstream sources);
- predict the oxygen balance of the polluted waters downstream from the waste sources; and
- predict the frequency with which wastewater inputs result in dissolved oxygen levels in the receiving body of water which exceed a wide range of DO values extending throughout the possible spectrum (say, 0 to 15 mg/l intervals of 0.5 mg/l).

Data for the study area are used to simulate the hypothetical response of the receiving water to the separate and combined effects of BOD waste inputs from: 1) upstream sources, 2) dry-weather urban sources, and 3) wet-weather urban sources. A system schematic is presented in Figure 1. The urban community served by a separate sewer system will convey stormwater runoff and municipal sewage through conduits which are not connected together. The BOD concentration of the storm sewer runoff is mixed with the dry-weather flow (DWF) and accumulated sewer solids. An interceptor carries the sanitary design flow to the municipal sewage treatment plant. The combined sewer overflow is either given treatment or allowed to discharge directly to the receiving water. Since complete mixing is assumed, the BOD concentrations of the combined sewer overflow (Q_c) and the flow (DWFCMB) intercepted for treatment by the DWF facility are identical. Any degree of treatment desired may be imposed at both the DWF and the wet-weather flow (WWF)

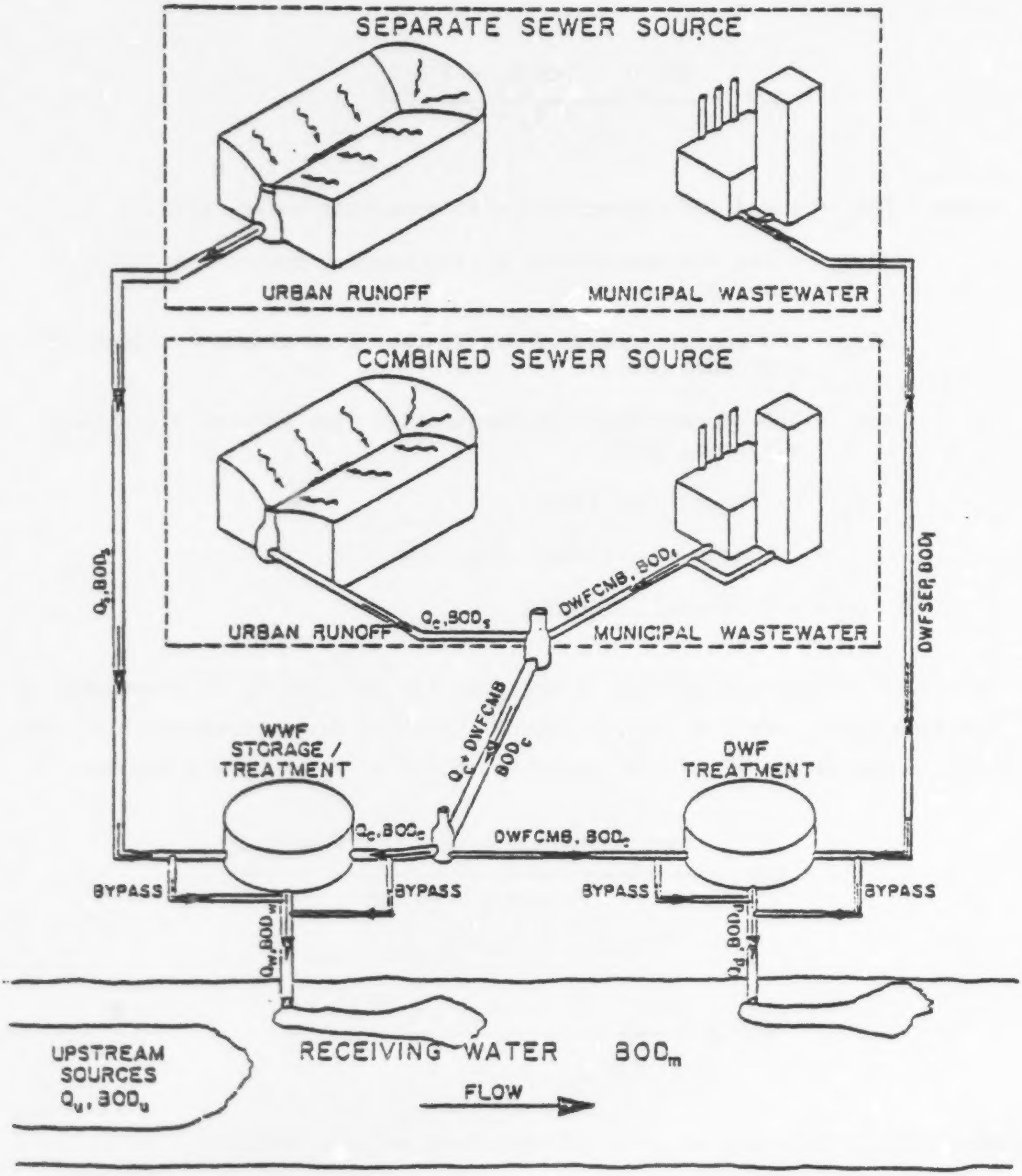
SEPARATE SEWER SOURCE
URBAN RUNOFF
MUNICIPAL WASTEWATER
COMBINED SEWER SOURCE
URBAN RUNOFF
Q_c, BOD_s
$DWFCMB, BOD_f$
MUNICIPAL WASTEWATER
Q_s, BOD_s
$DWFSER, BOD_f$
$Q_c + DWFCMB$
BOD_c
WWF STORAGE / TREATMENT
Q_c, BOD_c
$DWFCMB, BOD_c$
DWF TREATMENT
BYPASS
BYPASS
BYPASS
BYPASS
Q_w, BOD_w
Q_d, BOD_d
UPSTREAM SOURCES Q_u, BOD_u
RECEIVING WATER BOD_m
FLOW

Figure 1. Urban Wastewater Inputs to Receiving Body of Water

treatment plants. The concentration of the combined BOD inputs in the receiving water is given by:

$$BOD_m = \frac{BOD_u Q_u + BOD_d Q_d + BOD_w Q_w}{Q_u + Q_d + Q_w} \quad (1)$$

where BOD_m = mixed BOD concentration in receiving water, mg/l,

BOD_u = mixed BOD concentration from sources upstream of urban area, mg/l,

BOD_d = BOD concentration of dry-weather flow treatment plant effluent, mg/l,

BOD_w = BOD concentration of wet-weather flow treatment facility effluent, mg/l,

Q_u = upstream flow, cfs,

Q_d = DWF treated effluent, cfs, and

Q_w = WWF treated effluent, cfs.

The technique for calculation of the quantity and quality of stormwater and combined sewer overflows is discussed in further detail subsequently. The BOD concentrations of the DWF and WWF treated effluents are given by:

$$BOD_d = \frac{[BOD_f \cdot DWFSEP + BOD_c \cdot DWFCMB](1-R_d)}{DWFSEP + DWFCMB} \quad (2)$$

$$BOD_w = \frac{[BOD_s \cdot Q_s + BOD_c \cdot Q_c](1-R_w)}{Q_s + Q_c} \quad (3)$$

where BOD_f = BOD concentration of municipal sewage, mg/l,

BOD_c = mixed BOD concentration in the combined sewer, mg/l,

BOD_s = BOD concentration of urban stormwater runoff, mg/l,

DWFSEP = DWF contribution from separate sewer area, cfs,

DWFCMB = DWF contribution from combined sewer area, cfs,

Q_s = urban runoff carried by the separate storm sewer, cfs,

Q_c = combined sewer flow, cfs,

R_d = fraction removal of BOD achieved by the DWF treatment facility, and

R_w = fraction removal of BOD achieved by the WWF treatment facility.

The initial conditions of BOD in the river are defined by equation 1, and the hypothetical impact on the oxygen balance of the receiving stream is estimated by using simplified mathematical modeling approaches. The total hours of runoff-producing rainfall throughout the year are separated into storm events by defining a mininum interevent time. The procedure is discussed in detail subsequently. For a given storm event, the runoff and pollutant loads are summed and the critical DO deficit is estimated as a function of several stream parameters: temperature, flow, oxygen concentration, deoxygenation and reaeration rates, longitudinal dispersion, and BOD concentrations. The minimum DO is calculated subsequently and a frequency analysis is performed. Stream velocity is computed as a function of the discharge and the time and distance to each critical deficit point are obtained for each event.

The options used for the simulations include:

1. five inflow combinations:
   a. river flow + DWF
   b. river flow + DWF + separate flow
   c. river flow + DWF + combined flow
   d. river flow + separate flow + combined flow
   e. river flow + separate flow + combined flow + DWF,
2. four DWF treatment rates (variable),
3. three WWF treatment rates (variable), and
4. three fractions of measured upstream flow

may be investigated.

Item 4 is included as a model option to investigate whether the relative impact of urban stormwater runoff is most significant in the upstream portions of river basins. This effect may be simulated by simply reducing the upstream flow to any desired fraction of its actual measured value. Thus, discharge into a dry river bed may be studied.

CALCULATION OF URBAN RUNOFF QUANTITY AND QUALITY

The methods used to generate storm runoff flows and pollutant mass rates or concentrations depend upon the continuous hydrologic simulation model selected by the user. Such techniques are well documented for two models which may be applied to both urban and non-urban watersheds: 1) the Hydrologic Engineering Center's Storage, Treatment, Overflow, Runoff Model (STORM), and 2) the U. S. Environmental Protection Agency's Storm Water Management Model (SWMM), continuous version of Runoff Block. The urban runoff flows and associated pollutant concentrations and mass rates, derived from storm events over the drainage area of interest, represent concurrent time series which are read by the program through the standard card reader devices, or from peripheral storage units (disk/tape/drum). Since the program has the built-in capability of accessing a user-created data set, any continuous hydrologic and water quality model may be considered. The interfacing of Level III-Receiving with STORM and continuous SWMM is discussed in detail in the User's Manual (Medina, 1978). Regardless of the models used, it should be noted by the user that the urban runoff quantity and quality time series must represent hourly values.

PROGRAM OPERATION

The relationships among the main program and its subroutines are shown in Figure 2. The main program (hereafter referred to as subroutine MAIN) provides overall control and includes in its entirety the wet-weather flow model (WWFM). The WWFM is a mathematical abstraction of the physical system depicted earlier in Figure 1. The WWFM in subroutine MAIN simulates the conveyance system, including mixing in combined sewers of wet-weather and dry-weather pollutants during periods of runoff; the pollutant removal efficiency of various treatment schemes; mixing of the various pollutant inflow combinations with upstream sources in the receiving waters to determine initial conditions of BOD, DO, streamflow

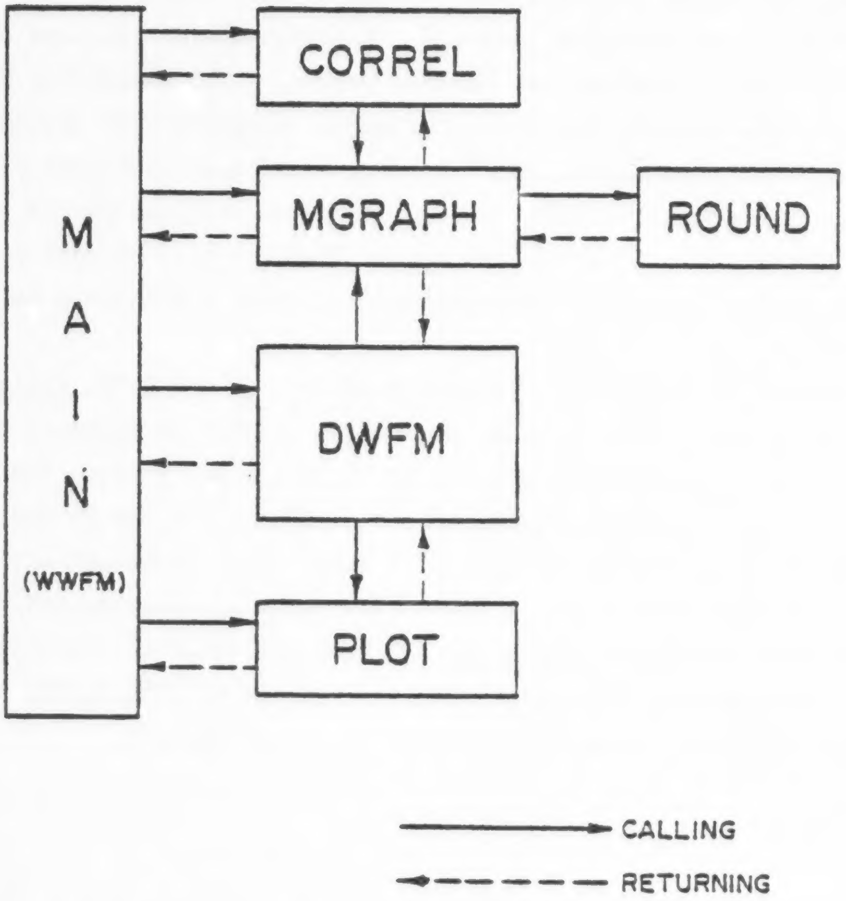
CORREL
MGRAPH
ROUND
M
A
I
N
(WWFM)
DWFM
PLOT
CALLING
RETURNING

Figure 2. Level III - Receiving Subprograms

and other parameters; and computes the oxygen balance of the polluted waters downstream from the waste sources. The procedure continues for each independent storm event as defined by the minimum intervent time (MIT). After all the wet-weather events have been thusly processed, frequency analyses are performed on either the resultant DO concentrations at a specified location downstream, or critical (minimum) DO concentrations as predicted by the model.

Subroutine CORREL subjects the hydrologic time series (either rainfall or runoff) to autocorrelation analysis. It automatically defines the MIT used in the WWFM to separate wet-weather events. This subroutine may be executed independently of the WWFM, but may be accessed only through subroutine MAIN. Subroutine MGRAPH, a single and multiple-curve plotting subprogram, is called by CORREL to display the correlogram of the time series. MGRAPH, in turn, calls subroutine ROUND to set the appropriate scale on the coordinates axes from examination of minimum and maximum values to be plotted.

Subroutine DWFM performs the same functions as the WWFM, during periods of no urban runoff. Thus, a model assumption is that no combined sewer overflows occur. Therefore, there is no "first-flush" effect, and no storm events over which to average pollutant loads. It may be executed independently of all other subroutines, except MAIN. Subroutine PLOT is called by the WWFM (contained in subroutine MAIN) and also by subroutine DWFM to display frequency histograms of receiving water DO concentrations, optionally. Likewise, subroutine MGRAPH is called by both models to plot cumulative, multiple frequency curves of DO concentrations.

COMPUTER SYSTEM CORE REQUIREMENTS

Level III-Receiving has been tested with two different Central Processing Units (CPU) and comparable supporting hardware. The Duke University Computation Center (DUCC) is connected by a high-speed microwave link to a dual IBM 370/165 configuration located at the Triangle Universities Computation Center (TUCC) in the Research Triangle Park. The Northeast Regional Data Center (NERDC), located at the University of Florida, is equipped with an AMDAHL 470-V6/II.

Level III-Receiving users have the option of running under either of the two standard IBM compilers, G or H. IBM's minimum system requirements for installation of the FORTRAN IV (G) compiler and the FORTRAN IV (H) compiler are: respectively, 128k bytes and 256K bytes of storage. Furthermore, the user may run Level III-Receiving from the actual card version (program source and data decks) or from a pre-compiled Load Module of the program stored on disk. All of these options result in varying core storage capacity requirements for the program. At TUCC, the FORTRAN G compiler is much faster and is recommended for all debugging runs. However, the FORTRAN H compiler has an optimization feature which results in the compiled coding being "optimized" for faster running in the execution step. Unless the user actually alters the program, debugging should be unnecessary and most errors will be due to incorrect formatting or sequencing of input data. Therefore, the program should be compiled in H and the coding stored on disk for future production runs. An additional feature is the IBM OS Loader, which replaces the Link-Edit and Go steps with a single, faster operation. Core storage capacity and average compilation times are presented in Table 2 for Level III-Receiving under TUCC's IBM 370/165 system. By comparison, the FORTRAN compiler of the AMDAHL 470-V6/II at NERDC required 110K bytes of core storage for execution.

Additional computer system requirements include peripheral storage devices which may consist of disk/tape/drum units depending upon machine configuration and user-selected input options.

METHODOLOGY

EVENT DEFINITION

The basic approach to define a wet-weather event is to analyze the runoff time series and establish the minimum number of consecutive dry-weather hours (DWH) that separates independent storm events. The independence of these events is not defined in a strictly climatologic sense, it is in fact statistically derived. The DWH refer to periods during which no runoff was produced. If STORM is selected to generate the

Table 2. Compilation Times[1] and Required Core Capacity[2]

Compiler	Compile Time,sec	Link-Edit Time,sec	Compile Core bytes	Execute Core bytes
FORTRAN G	29.4	10.3	146K	104K
FORTRAN H	60.3	9.9	300K	100K
FORTRAN G with IBM OS Loader	28.6	none	146K	200K

[1] Average values.

[2] IBM 370/165 system, Triangle Universities Computation Center.

hydrologic time series, depression storage and evaporation rates must be satisfied before any runoff is predicted but no runoff occurs during periods of no measurable precipitation. The continuous version of SWMM allows runoff to decay temporally beyond intervals with zero precipitation input. Therefore, for an identical precipitation time series, runoff events generated by SWMM will generally be of longer duration.

The runoff time series is subjected to autocorrelation analysis. For hydrologic processes, it is practical to estimate the autocorrelation coefficients by an open-series approach (Yevjevich, 1972 and Fiering and Jackson, 1971):

$$r_I(k) = \frac{\sum_{i=1}^{n-k} x_i x_{i+k} - \frac{1}{n-k}\left[\sum_{i=1}^{n-k} x_i\right]\left[\sum_{i=k+1}^{n} x_i\right]}{\left[\sum_{i=1}^{n-k} x_i^2 - \frac{1}{n-k}\left(\sum_{i=1}^{n-k} x_i\right)^2\right]^{0.5}\left[\sum_{i=k+1}^{n} x_i^2 - \frac{1}{n-k}\left(\sum_{i=k+1}^{n} x_i\right)^2\right]^{0.5}} \qquad (4)$$

where $r_I(k)$ = sample estimate of lag-k autocorrelation coefficient for hydrologic process I,

x_i = discrete data series (observations) of hydrologic process I, for $i = 1,2,\ldots,n$,

n = total number of data points or observations, and

k = number of hourly lags.

The tolerance limits for a normal random time series which is circular and of lag 1 is given by (Anderson, 1942):

$$\mathrm{TL}\ [r_I(1)] = \frac{-1 \pm t_\alpha \sqrt{n-2}}{n-1} \qquad (5)$$

where

t_α = standardized normal variate corresponding to probability level $1 - \alpha$.

A circular time series is defined as a series where the last value is followed by the first so that the time series repeats itself. Equation (5) has been extended for use with an open series, for the general lag case (Yevjevich, 1972). At a 95 percent probability level, the tolerance limits are given by:

$$\mathrm{TL}\ [r_I(k)] = \frac{-1 \pm 1.645\sqrt{n-k-1}}{n-k} \tag{6}$$

A plot of the serial correlation coefficients, $r(k)$, against the number of lags, k, is called a correlogram. The technique of autocorrelation analysis is essentially a study of the behavior of the correlogram of the process under investigation (Quimpo, 1968). The model compares the value of $r(k)$ obtained from equation (1) with $\mathrm{TL}\ [r_I(k)]$, computed by equation (3), for the corresponding number of hourly lags k. The minimum inter-event time (MIT) which separates independent wet-weather events is defined as the minimum value of k for which $r(k)$ is not significantly different from zero at a 95 percent probability level.

EFFECT ON RECEIVING WATERS

A simplified mathematical modeling approach is used in which critical deficits and resulting minimum DO concentrations are determined for a large number of waste input combinations, treatment schemes, and receiving water conditions. The development of a detailed and sophisticated model is not justified for the problem context: to provide adequate information on the relative effectiveness of various pollutant control strategies in achieving selected water quality standards. The basic theory of mathematical modeling of one-dimensional bodies of water is presented for the spectrum of natural systems from freshwater streams to tidal rivers and estuaries. The approach is particularly advantageous for a limited data base on natural system geometry, hydrodynamic variables, and discrete rather than continuous water quality measurements.

Assumptions typical of models limited for interim planning are made (Hydroscience, Inc. 1971):

(1) Temporal steady-state conditions prevail, where all system parameters and inputs are constant with respect to time, however, a relatively short time step (1 hour) is used for simulation.

(2) Natural system parameters (such as flow, velocity, hydraulic depth, deoxygenation and reaeration rates, and longitudinal dispersion) are spatially constant along the flow axis throughout each time step.

(3) All waste inflows to the receiving body of water occur at one point.

(4) The effects of various natural biological processes (algal photosynthesis and respiration, benthal stabilization) are incorporated into a background quality which is reflected by DO deficit (if none, by saturation) upstream from the waste inflow point. Any benthic buildup is incorporated into the BOD decay rate.

(5) Waste treatment facilities operate at constant efficiencies, independent of hydraulic and organic loadings, for the entire period of simulation.

Oxygen Balance of Polluted Streams and Estuaries

In view of the modeling objectives, pollutant transport processes in these systems may be adequately approximated by the one-dimensional version of the classical convective diffusion equation. This partial differential equation is based on the principle of conservation of mass (continuity) and is given by:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x} \left[E\frac{\partial C}{\partial x} - UC \right] \pm \Sigma S \qquad (7)$$

where C = concentration of water quality parameter (pollutant), M/L^3,

t = time, T,

$-E\frac{\partial C}{\partial x}$ = mass flux due to longitudinal dispersion along the flow axis, the x direction, M/L^2T,

UC = mass flux due to advection by the fluid containing the mass of pollutant, M/L^2T,

S = sources or sinks of the substance C, M/L^3T,

U = flow velocity, L/T, and

E = longitudinal dispersion coefficient, L^2/T.

The equation assumes no diffusion of pollutants through the natural body of water boundaries (other than what may be included in the source-sink term) and is best suited to predict concentrations relatively far downstream from the point of waste injection. Since critical DO deficits usually occur some distance downstream from the waste source, equation (7) is particularly well suited for such predictions.

APPLICATION TO STUDY AREA

The City of Des Moines, Iowa is located near the confluence of the Des Moines River and the Raccoon River. It contains approximately 200,000 people out of the total of 288,000 for the metropolitan area (Davis and Borchardt, 1974). The mean annual precipitation is 31.27 inches (795 mm) which is approximately equal to the United States average, and the average value for the State of Iowa. The urban area covers 49,000 acres (19,830 ha) of land which has gently rolling terrain. Most of the area, 45,000 acres, is served by separate sewers, while 4,000 acres are served by combined sewers.

Selection of the study area was based primarily on data availability. Henningson, Durham & Richardson, Inc., Omaha, Nebraska, conducted an extensive sampling program of combined sewer overflows, stormwater discharges, and surface waters in the Metropolitan Des Moines for the U. S. Environmental

Protection Agency (Davis and Borchardt, 1974). The objective was a combined sewer overflow abatement plan. The sampling program was conducted from March 1968 to October 1969. Other considerations revolved around the fact that Des Moines, Iowa is somewhat typical of many urban centers throughout the country:

(1) it has a medium-sized population;

(2) its domestic and industrial dry weather flows receive secondary treatment;

(3) its wastewaters are discharged into a non-tidal receiving stream; and

(4) the urban area receives a mean annual precipitation equal to the national average.

The verification procedure was preceded by calibration of the urban runoff BOD_5 loading rates for Des Moines, Iowa, as computed by STORM. The dust and dirt surface loading factors were adjusted to obtain an annual average BOD_5 concentration of 53 mg/l for urban stormwater runoff. The above concentration was the average value determined by the field monitoring program in the separate sewer system. The developed mathematical model, as discussed in the overview, simulates the mixing of stormwater runoff and sanitary sewage in the combined sewer system. The annual average BOD_5 concentration of combined sewer overflows was computed to be 75 mg/l, including the effects of first flush. The average value measured in the combined sewer system was reported to be 72 mg/l. Model parameters were adjusted to obtain an adequate fit between calculated and observed profiles of dissolved oxygen. These curves are shown in Figure 3, along with other pertinent information, and correspond to a point 5.6 mi (9.0 km) downstream from the confluence of the Raccoon and Des Moines Rivers.

The autocorrelation function of hourly runoff events for Des Moines, Iowa is presented in Figure 4. At a number of hourly lags equal to zero, the correlation of the discrete open series is unity because this point represents the linear dependence of the data series on itself. A significantly zero correlation (within 95 tolerance limits) between runoff events first occurs at a lag of approximately 9 hours, defining the minimum interevent time. The physical interpretation is that periods without runoff for at least 9 hours separate uncorrelated, and therefore independent, wet weather events. The hourly urban urban runoff and associated pollutant

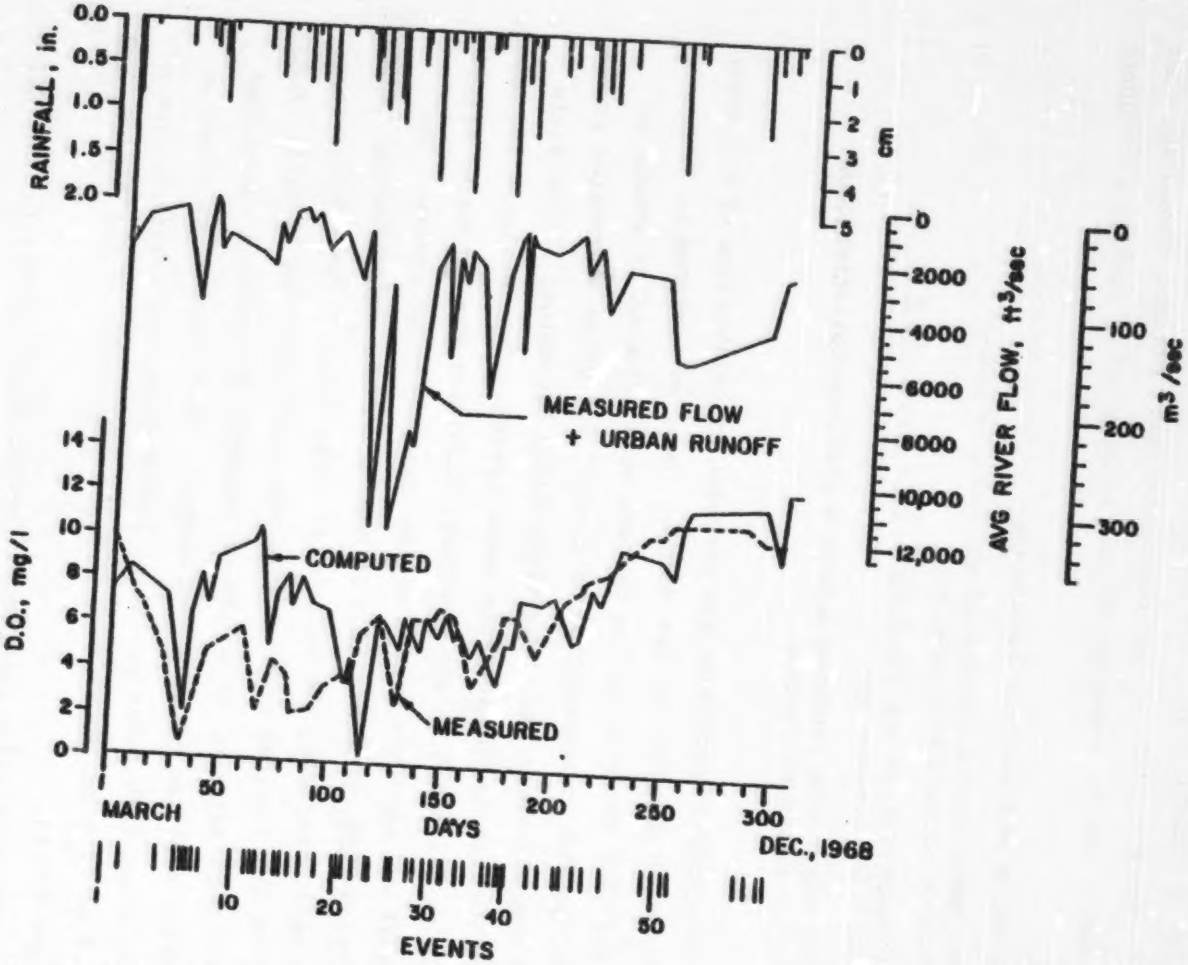

Figure 3. Application to Des Moines, Iowa. Measured and computed values of DO at 5.6 mi (9.0 km) downstream from confluence of Raccoon and Des Moines Rivers.

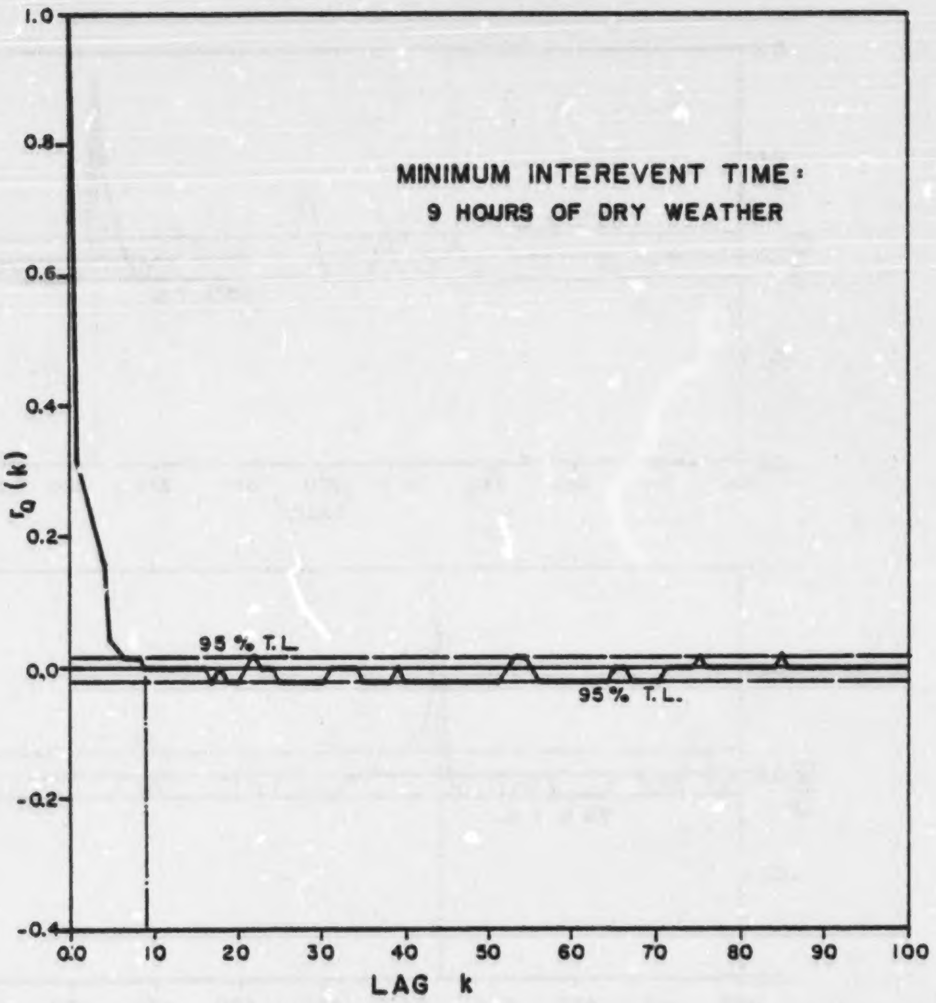

Figure 4 Autocorrelation Functions of Hourly Urban Runoff for Des Moines, Iowa, 1968.

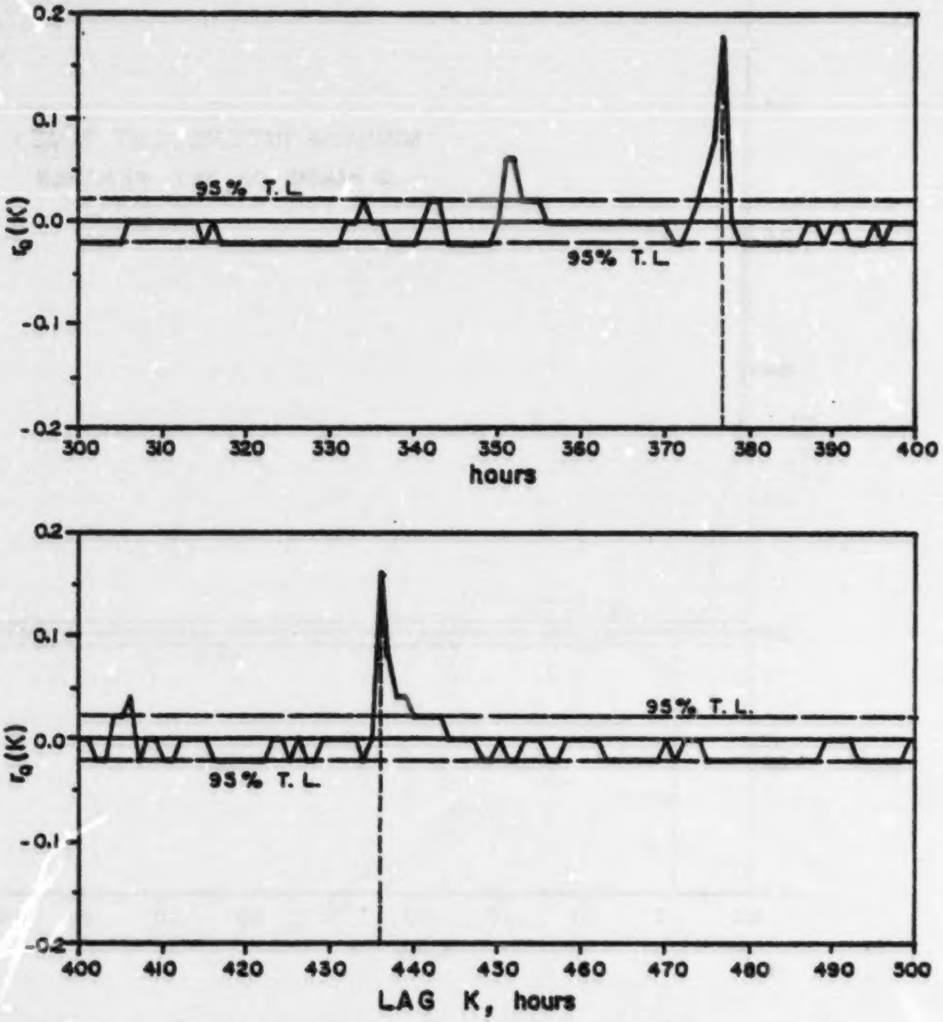

Figure 4 (continued) - Autocorrelation Function of Hourly Urban Runoff for Des Moines, Iowa, 1968.

loads within each event (including DWF pollutant loads during DWH periods less than nine hours duration) are summed, average conditions are determined, and the model proceeds with the receiving water analysis.

Based on precipitation records obtained from the NOAA Environmental Data Service (Asheville, North Carolina), the total rainfall that fell over Des Moines, Iowa, during 1968 was 27.59 inches (701 mm). STORM computed a total runoff of 10.28 inches (261 mm) over a water shed area of 49,000 acres (19,600 ha), for an overall urban area runoff coefficient of 0.37. There were 65 days in the year during which rainfall was recorded, from which 58 wet weather events were defined. The results are presented in the form of minimum DO frequency curves for the wet weather and dry weather periods throughout the calendar year.

It is appropriate to examine the model estimates of critical DO concentration in the Des Moines River, for all waste inputs, for conditions assumed to exist in 1968 during periods of urban runoff: (1) secondary treatment (85 percent BOD removal) of DWF; (2) no stormwater treatment; (3) full river flow (100 percent of measured flow); and (4) combined sewer area 8.16 percent of the total urban area. Figure 5 represents the minimum DO frequency curves for these conditions. The curves indicate clearly that all combinations including a substantial amount of wet weather flow (WWF) result in a drastic decrease in river minimum DO concentrations. For example, 42 percent of all the wet weather events throughout the year produced conditions in the receiving water that caused minimum DO levels below 4.0 mg/l. Combined sewers contributed WWF from only 8 percent of the total urban area modeled, yet the BOD_c concentration was sufficiently high to inflict an appreciable reduction in DO levels when compared to DWF sources during periods of runoff.

Figure 6 displays the minimum DO frequency curves obtained by varying the percent of the total urban area served by combined sewers. There is a substantial, but not drastic, decrease in water quality when the extreme conditions are compared: an area served only by separate sewers (0 percent combined) versus an area served exclusively by combined sewers. The curves support the theory that total separation of sewers is not the answer to the control of urban runoff pollution. The curves in this figure all represent secondary treatment of DWF, no urban runoff

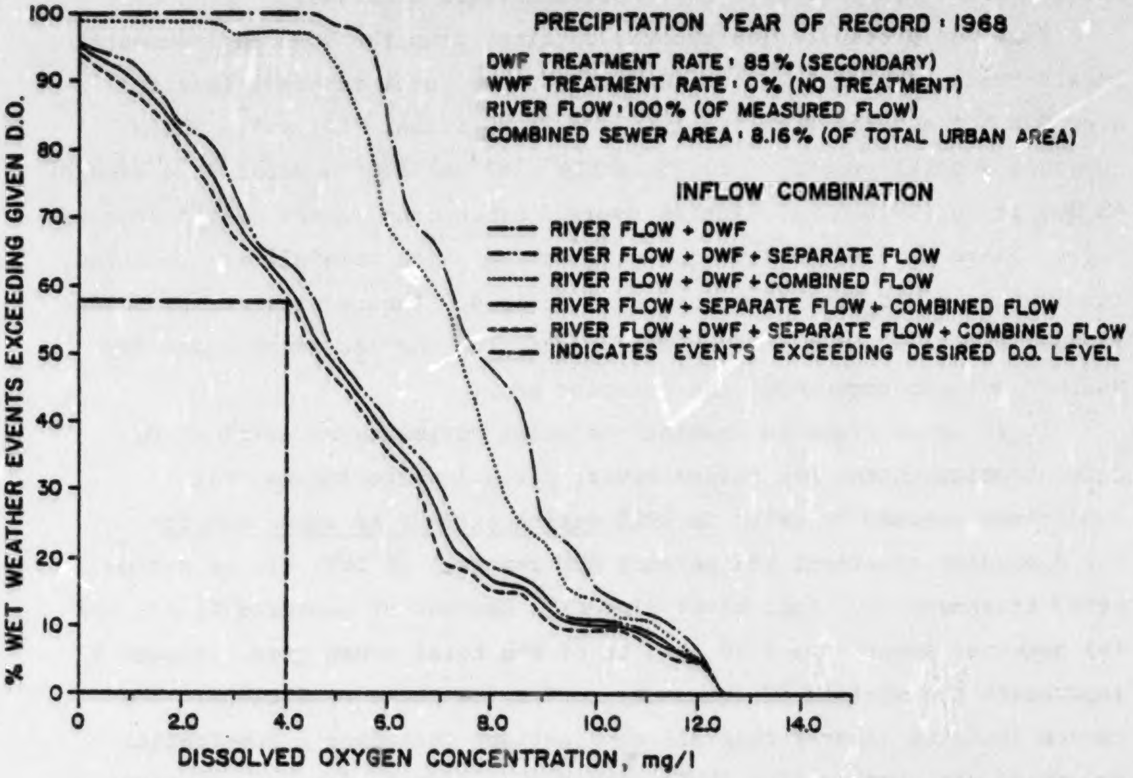

Figure 5. **Minimum DO Frequency Curves for Existing Conditions in the Des Moines River.**

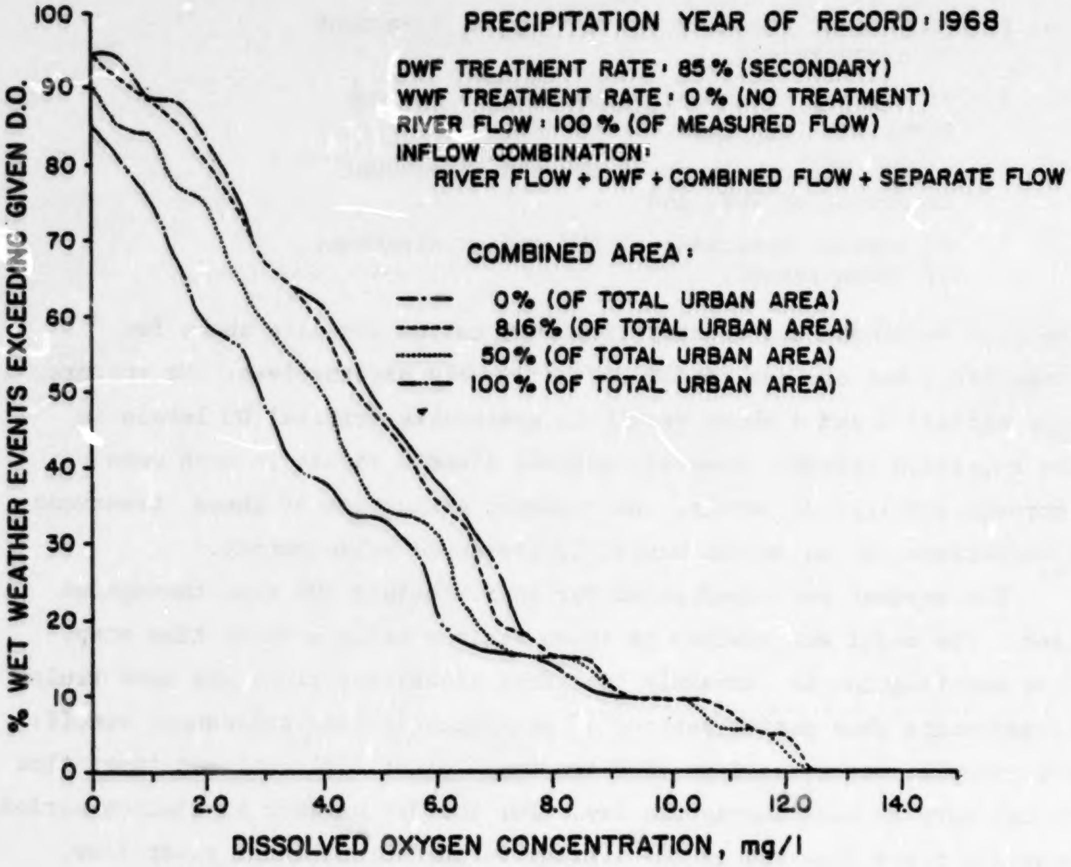

Figure 6.- Minimum DO Frequency Curves for Varied Percent of Combined Sewer Area.

treatment, and full river flow.

The minimum DO frequency curves in Figure 7 compare four treatment alternatives to reduce water pollution during periods of urban runoff:

1. 95 percent treatment of DWF and no treatment of urban runoff,
2. 85 percent treatment of DWF and 25 percent treatment (BOD removal) of WWF,
3. 85 percent treatment of DWF and 75 percent treatment of WWF, and
4. 85 percent treatment of DWF and no treatment of urban runoff.

The zero treatment and primary treatment curves are also shown for comparison, but are not considered acceptable alternatives. It appears that options 1 and 4 above result in comparable critical DO levels in the receiving stream. However, options 2 and 3 result in much more improved critical DO levels. An economic evaluation of these treatment alternatives, on an annual basis, is presented subsequently.

Dry weather was experienced for approximately 300 days throughout 1968. The model was applied to these periods using a daily time step. This modification is certainly justified since conditions are more truly steady-state than during periods of precipitation and subsequent runoff: for example, waste loadings (DWF treatment plant effluent) and river flow do not vary as much during the day. For the dry weather simulation period, upstream river flow was on the average 94 percent of total river flow, ranging from 82 percent to 99.6 percent. The results are shown in Figure 8. A remarkable 97 percent of the dry weather days exceed a minimum DO concentration of 4.0 mg/l. Upgrading DWF treatment becomes meaningful only if stream DO standards are set higher than 4.0 mg/l. The Des Moines River carries a relatively high BOD load upstream of the Des Moines urban area. Thus, even during dry weather periods only, a significant increase in the DWF treatment rate does not result in a corresponding increase in the critical DO levels.

To maintain the proper perspective, it is desirable to view the effects of urban runoff on an annual basis, not just during periods of wet weather. The frequency curves shown in Figure 7 and Figure 8 are combined by weighting on the basis of the number of rainfall days and dry weather days in the year. The composite totals are presented in

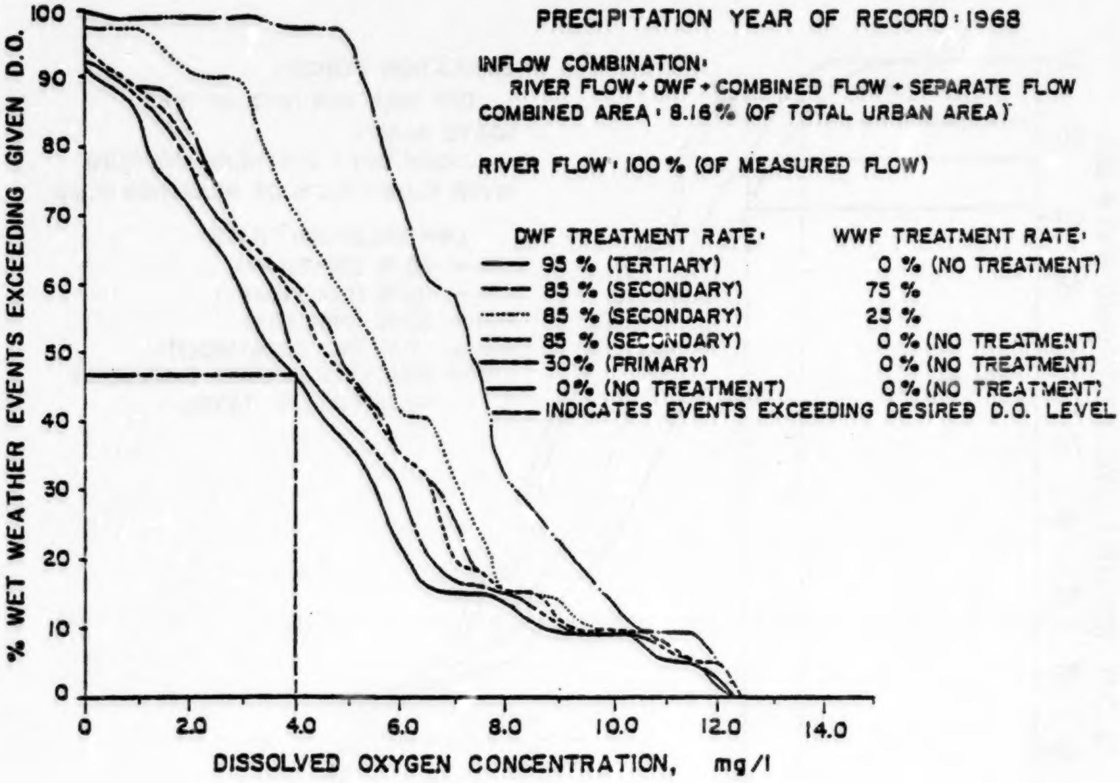

Figure 7. Minimum DO Frequency Curves for Varied Treatment Alternatives.

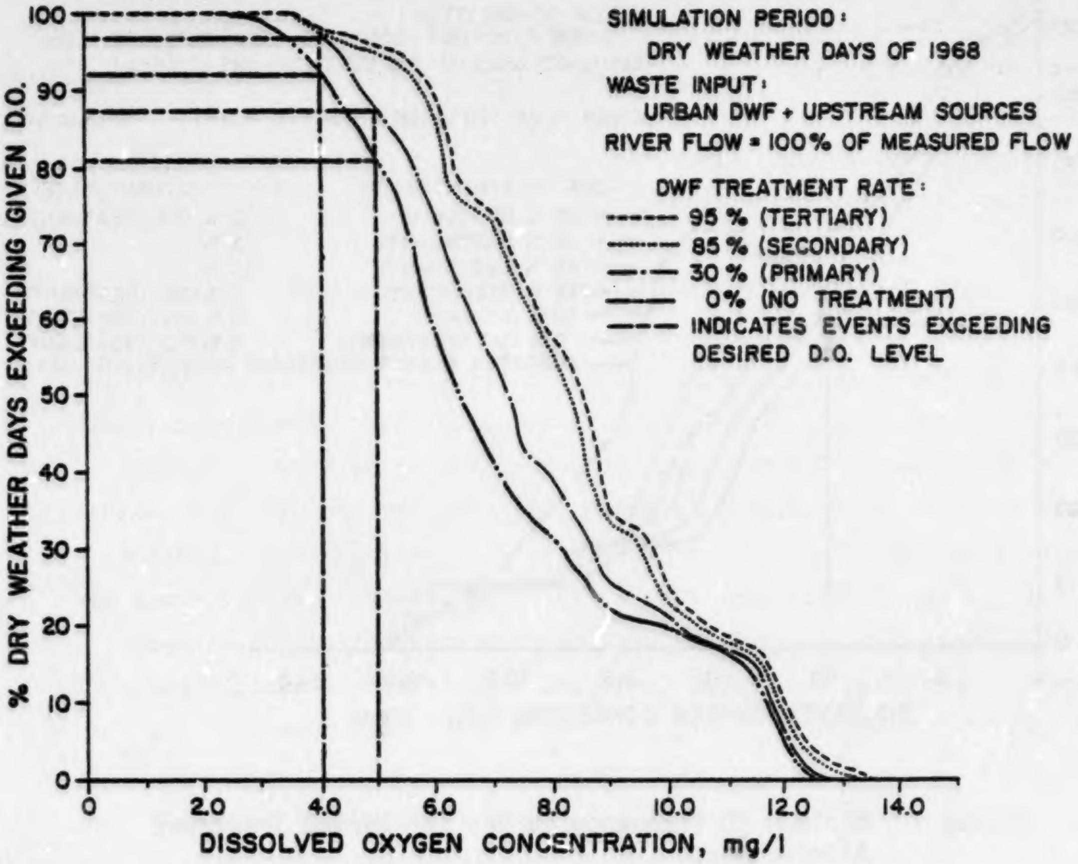

Figure 8. Dry Weather Minimum DO Frequency Curves for Varied DWF Treatment Alternatives.

Figure 9. For example, a given stream standard of 4.0 mg/l is exceeded 90 percent of the time for existing conditions in Des Moines, Iowa, throughout the year 1968. A significant amount of treatment (75% BOD removal) of WWF in addition to secondary treatment of DWF results in critical DO levels such that the same stream standard is exceeded 97 percent of the days in the year. Annual DO duration curves tend to mask the impact of shock loads of organic pollutants discharged during periods of urban runoff. A few extended violations of stream DO standards may cause anaerobic conditions resulting in fish kills and proliferation of undersirable microorganisms.

MODEL UTILITY FOR AREAWIDE WATER QUALITY MANAGEMENT PLANS

Municipal water pollution control alternatives should be evaluated in terms of pollutant removal efficiency, receiving water impacts, and associated costs. The true cost-effectiveness of various treatment strategies can be determined realistically only by a continuous analysis of the frequency of violation of established water quality standards. Of course, in the selection of the best control strategy other factors may become important: 1) recovery of receiving waters from shock loads generated by urban runoff, 2) local and regional water quality goals in addition to established standards, 3) public willingness to pay the additional costs associated with increased control, and 4) dual use of WWF facilities, as DWF treatment units during periods of no runoff.

The cost figures shown in Table 3 represent the additional expense incurred in providing storage/treatment beyond that already available with secondary treatment of DWF and no control of urban runoff (existing conditions for Des Moines, Iowa). Details of the cost assessment for typical wet-weather and dry-weather control facilities have been presented in the nationwide study (Heaney, et al., 1977). The Des Moines River stretches for 200 mi (322 km) from the City of Des Moines to its junction with the Mississippi River and is generally wide and swift with broad flood plain. Bottom material is composed of silt deposits, sand, gravel and rubble providing numerous habitats for fish and other aquatic life (State Hygienic Laboratory, 1974). The entire reach is classified by the Iowa Water Quality Standards such that the absolute minimum DO level

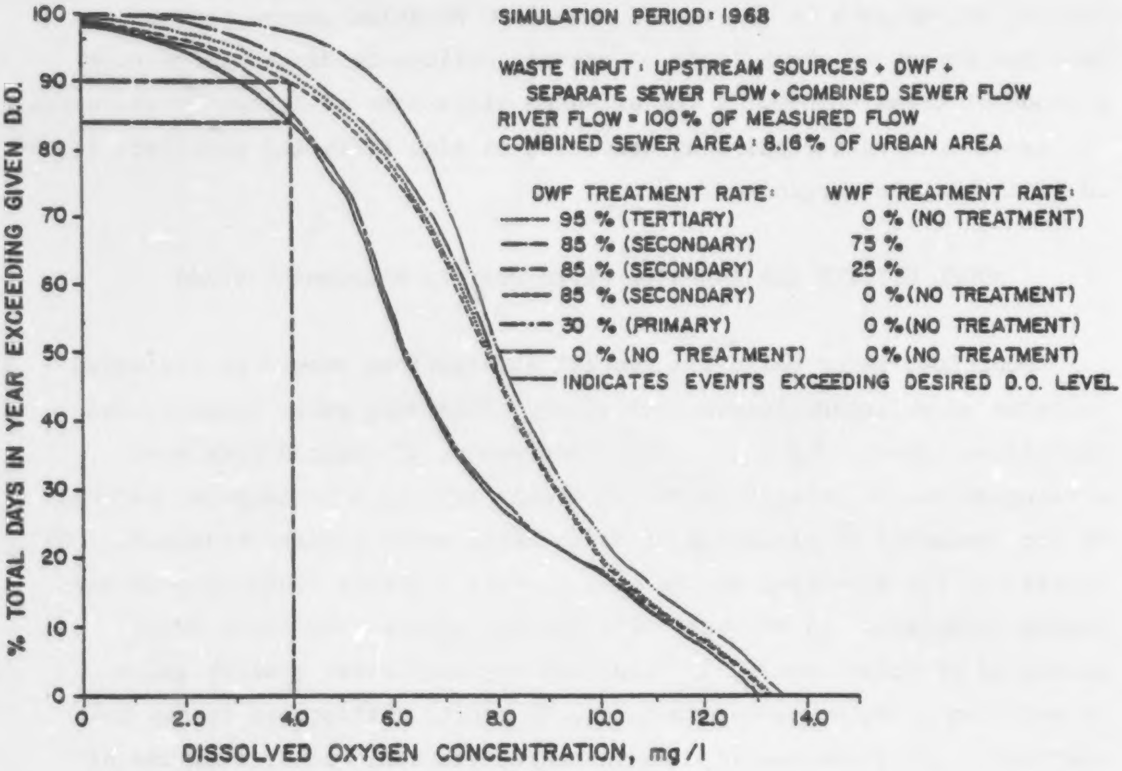

Figure 9. Annual Minimum DO Frequency Curves.

Table 3. DWF Tertiary Treatment vs. WWF Control[1]
(Heaney, et al., 1977)

Options	Amortized Annual Capital Cost $ (20 yrs, 8%)	Operation and Maintenance Cost ($/yr)	Total Annual Cost ($/yr)
1. DWF Complete Tertiary Treatment, No WWF Treatment	2,158,000	4,132,000	6,290,000
2. DWF Activated Sludge-Coagulation-Filtration, No WWF Treatment	---	---	1,664,000
3. WWF 75% BOD Removal, DWF Secondary Treatment	---	---	9,293,000
4. WWF 25% BOD Removal, DWF Secondary Treatment	---	---	816,000

---

[1] Based on 49,000 acres (19,600 ha) of developed urban area with population approximately 200,000. The total annual cost includes amortized capital cost (20 yrs, 8%) and operation and maintenance costs.

must equal 4.0 mg/l, and 5.0 mg/l during at least 16 hours per day (State Hygienic Laboratory, 1970). The effects of various control strategies upon the critical (minimum) DO concentrations of the Des Moines River have been presented. Thus, taking 4.0 mg/l as the standard or basis for water quality comparisons, the different control options may be judged by the following criteria:

1. total annual cost, and
2. violations of the minimum allowable dissolved oxygen level.

Table 4 summarizes control costs versus DO standard violations for two advanced waste treatment options, two wet-weather control options, and existing DWF secondary treatment facilities. For comparative purposes, two additional treatment conditions which are not presently acceptable by government regulation are present. Detailed process flow charts for the DWF facilities are included in the nationwide assessment (Heaney, *et al.*, 1977). Results of the simulation and the economic evaluation reveal that:

1. since both types of tertiary treatment remove essentially the same amount of BOD_5, option 1 is justified over option 2 only when nutrient removal is necessary;
2. option 4 is preferred over any form of advanced waste treatment;
3. option 3 is attractive because it causes the least amount of damage to the receiving stream, but it is the most expensive alternative; and
4. any reduction in the degree of DWF treatment for existing conditions, option 5, results in a substantial deterioration to receiving water dissolved oxygen levels and must be weighed against the savings incurred.

Furthermore, the issue of shock load prevention seems to favor high levels of WWF control.

The user should be cautioned that the above results were derived from application of the model to a specific urban area for a historical record of storm events occurring during a particular year. However, the usefulness of the methodology applied should be clear. The success of its application still relies heavily on the quality of the field data available. Dissolved oxygen in the receiving water body has been used as the key indicator of water quality, yet other parameters may have to be considered depending upon water quality goals. Complex hydrodynamic

Table 4. Cost-Effectiveness of Control Options

Options	% Wet-Weather Events[1] Violating Standard	% Dry Days in Year Violating Standard	Total Incremental Annual Cost[2] ($/yr)	Total No. of Days During Year that Standard is Violated[3]
1. DWF Complete Tertiary Treatment, No WWF Treatment	40	1.5	6,290,000	31
2. DWF Activated Sludge Coagulation-Filtration Treatment, No WWF Treatment	40	1.5	1,664,000	31
3. DWF Secondary Treatment WWF 75% BOD Removal	3	2.0	9,293,000	8
4. DWF Secondary Treatment, WWF 25% BOD Removal	30	2.0	816,000	26
5. DWF Secondary Treatment, No WWF Treatment	42	2.0	0	33
6. DWF Primary Treatment[4], No WWF Treatment	50	3.0	-1,438,000	42
7. No DWF Treatment[5], No WWF Treatment	53	7.0	-1,843,000	55

---

[1] Defined by a minimum interevent time of 9 DWH.

[2] In addition to control costs for existing conditions (option 5).

[3] Based on a minimum allowable DO concentration of 4,0 mg/l.

[4] Savings incurred by reducing DWF treatment of trickling filter plant of 35.3 mgd (1.55 cu m/sec).

[5] Savings by completely eliminating treatment.

conditions will require additional, more detailed modeling efforts. Engineering judgment must be exercised carefully in the interpretation of model output and the importance of verification procedures cannot be overemphasized. The model is a user assistance tool in the preparation of water quality management plans, not a decision-maker by itself.

CONCLUSIONS

A simplified continuous receiving water quality model has been developed to permit preliminary planning and screening of areawide urban wastewater treatment alternatives in terms of frequency of water quality violations.

Model Capabilities

1. Level III-Receiving may be interfaced through peripheral storage devices with various hourly, continuous urban catchment hydrologic simulation models.
2. Continuous analysis of receiving water quality allows representation of the impacts due to the random occurrence and probabilistic nature of hydrologic phenomena.
3. A large number of wastewater inflow combinations to the receiving body of water, dry-weather flow and wet-weather flow treatment rates, and upstream flow conditions may be simulated.

Model Applications

1. Level III-Receiving has been developed on a general basis so that it may be applied to any urban drainage basin by simply changing the input data to reflect the particular study area and hydrologic time series. There is virtually no limitation to the size of catchment modeled.
2. In theory, an unlimited number of storm events may be processed; however, practical considerations such as computer time and costs may be limiting to some users.

3. Data requirements are common to engineering analysis on non-point source problems and complete instructions on data preparation are provided.
4. Field measurements, quantitative and qualitative, are necessary to adequately calibrate model parameters and verify predicted values.

Model Limitations

1. The methodology is not applicable to stream and estuarine systems exhibiting complex hydrodynamic characteristics, or of such geometry that multi-dimensional analysis is required.
2. Complex water quality conditions, such as eutrophication, non-linear kinetic interactions, sedimentation and sediment exchange are not accounted for by the mathematical representation of the physical system.

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"USE OF THE HYDROLOGICAL ENGINEERING CENTER STORAGE, TREATMENT, OVERFLOW, RUNOFF MODEL - 'STORM' FOR SEWER INFLOW REMOVAL/TREATMENT COST EFFECTIVE ANALYSES IN THREE SELECTED LOCALITIES"

Rodney T. Prosser, Project Engineer
Howard M. Shapiro, Director of Engineering
LOZIER Architects Engineers

Rochester, New York

INTRODUCTION TO'INFLOW' & 'INFILTRATION'

Extraneous undesirable flows reaching a sanitary sewer system are generally classified into two categories, 'Inflow' and 'Infiltration'. It is generally accepted that infiltration is caused by leakage from defective pipe and manholes during high groundwater levels and that inflow is the product of direct stormwater runoff connections to the sanitary sewer system. 'Infiltration' is generally a relatively constant, seasonally varying flow and 'Inflow' a rapidly fluctuating, shorter and more intense flow immediately following rainfall occurrences. When evaluating sanitary sewer systems the distinction is not generally so clear as the definitions would suggest and to differentiate the two extraneous flow causes may be nearly impossible. In fact, the same defective sewer joint may be a source of both 'Inflow' from direct runoff during rainy periods and 'Infiltration' during the Spring if the groundwater level is high and possibly both at once.

Because 'Inflow' is generally a severe, peaking flow condition, heavily taxing the capacity of both sewers and treatment facilities, and because it is very difficult to estimate what its ramifications might be for various intensity-duration rainfalls over a period of several years, a reasonable approach for evaluating its probable impacts and related system costs had to be found.

In order to compare the costs of removal of inflow to treatment of inflow a realistic estimate of the amount of inflow to be expected over a design period must be established. Inflow characteristics are very similar to stormwater runoff flow characteristics, except for a decreased quantity (to be expected) and increased rainfall/inflow lag time. Both these characteristics can be accomodated by a simple 'stochastic' model incorporating runoff coefficients of imperviousness and a method for placing runoff volume into its appropriate time interval, i.e. the unit hydrograph.

The H.E.C. model 'STORM', was chosen for the analyses. It is a 'long-term' duration model that simply, but effectively meets all the necessary requirements. The simplistic nature of the model requires that judicious engineering judgement be used to set up appropriate system calibration factors based on actual physical system characteristics. In most cases, this can be adequately accomplished after flow monitoring of various selected rainstorms. User's experience and ability to understand the real mechanics allow meaningful results to be obtained from a simple model of a highly complex situation. More detailed models of a parametric (more closely approximated) and deterministic nature (function for each relationship of importance) were not required by the needs of this study. A close statistical probability and frequency of various storm related excess flow occurrences was needed and the capabilities of 'STORM' for meaningful long term statistical and summary analysis made it ideally suited for the application. Though its accuracy might be questionable for close agreement of instanteous rainfall and related inflow rates in any one storm occurrence, the total volumes of inflow predicted based on many storms was found to agree very closely with those measured and generally this was all that was needed in the cost analyses.

'STORM' provides system analysis as depicted in Fig. Nc. I. Rainfall striking impervious areas becomes runoff after initial abstractions (mostly for depression storage) have been satisfied. The runoff will become inflow if tied into the sewer system and must subsequently be treated or, in the case of a conventional combined system, possibility overflow the system. The relationship of rainfall

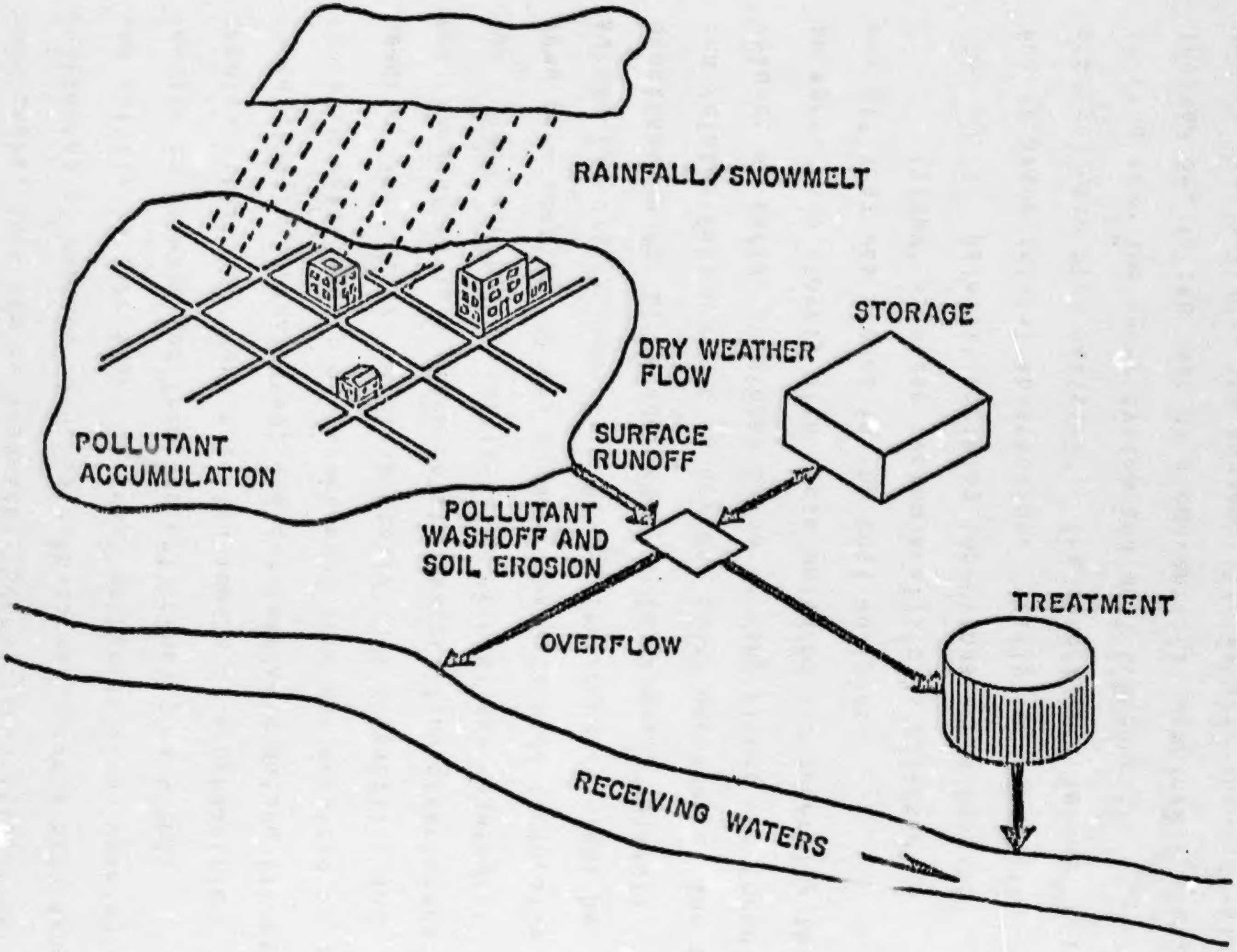

RAINFALL/SNOWMELT
POLLUTANT ACCUMULATION
DRY WEATHER FLOW
STORAGE
SURFACE RUNOFF
POLLUTANT WASHOFF AND SOIL EROSION
TREATMENT
OVERFLOW
RECEIVING WATERS

Figure 1. MAJOR PROCESSES MODELLED BY STORM

to sewer inflow is governed by initial abstractions, imperviousness factors, drainage factors and time of travel defined by the SCS unit hydrograph; no sewer routing techniques are employed. All inflow as well as base sanitary flow will first go to treatment until capacity is exceeded, then subsequently through storage and to overflow; stored flow is bled through treatment when allowable. All flow eventually discharges to the receiving water.

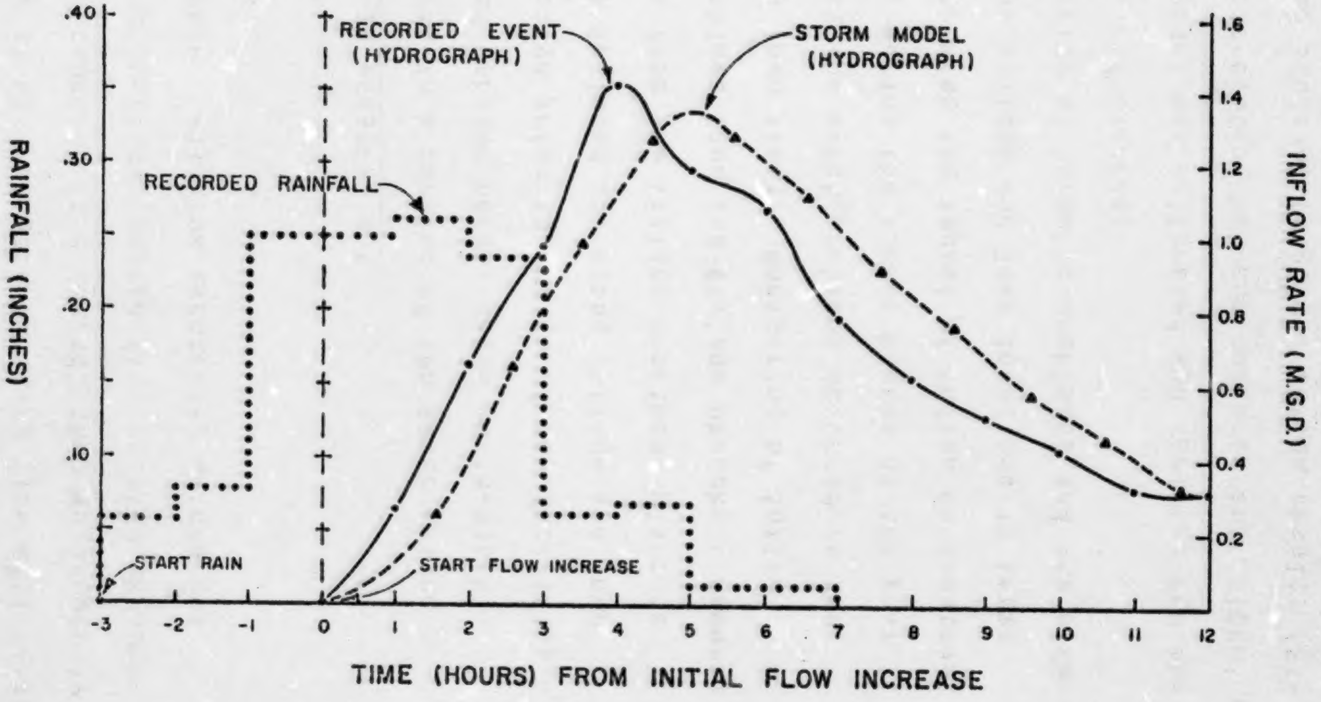
INFLOW INVESTIGATION IN WEBSTER, N.Y.

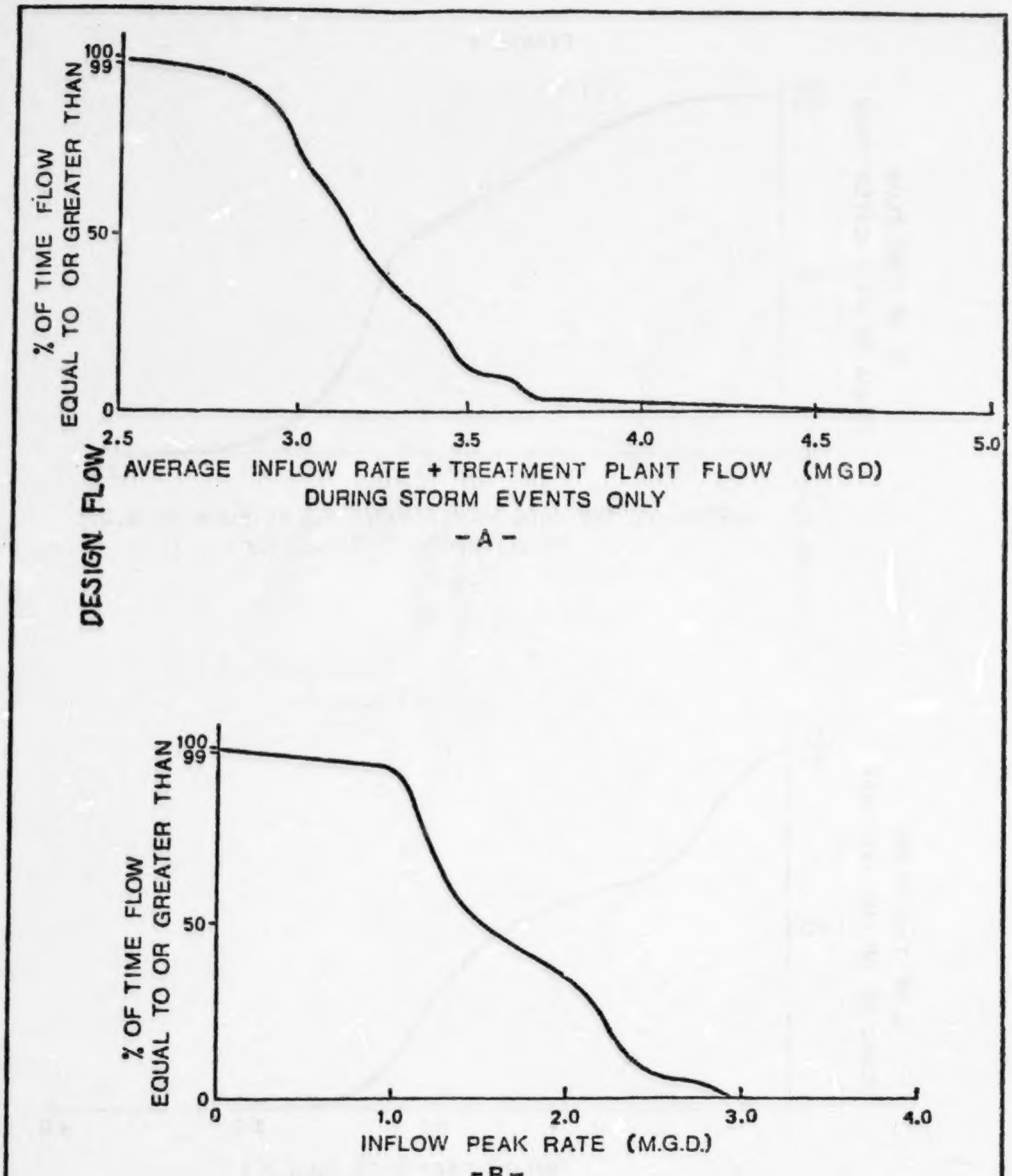
Inflow problems in a portion of the sewer system serving the Village of Webster necessitated an in-depth analysis of the situation prior to design of new facilities. Proposed improvements included upgrading Village and Town facilities. Effluent from the Village treatment plant is to be diverted to combined Town and Village phosphate removal facilities at the new Town plant. Monitoring of inflow clearly indicated that the worst problems occurred in the older Village system, during the summer months of the year. It was desired to determine the impact of inflow on treatment facilities at both the Village and Town locations in terms of frequency and severity of hyraulic overloads and the effect on treatment facility efficiencies.

The 'STORM' model was calibrated and verified for the system. Very close correlation between measured and 'STORM' generated inflow volume was achieved (Fig. 2). Inflow results (Fig.'s 3 & 4) for the Village and Town plants indicated that even peak

FIGURE 2

VILLAGE OF WEBSTER
INFLOW VOLUME VERIFICATION
APRIL 22, 1977





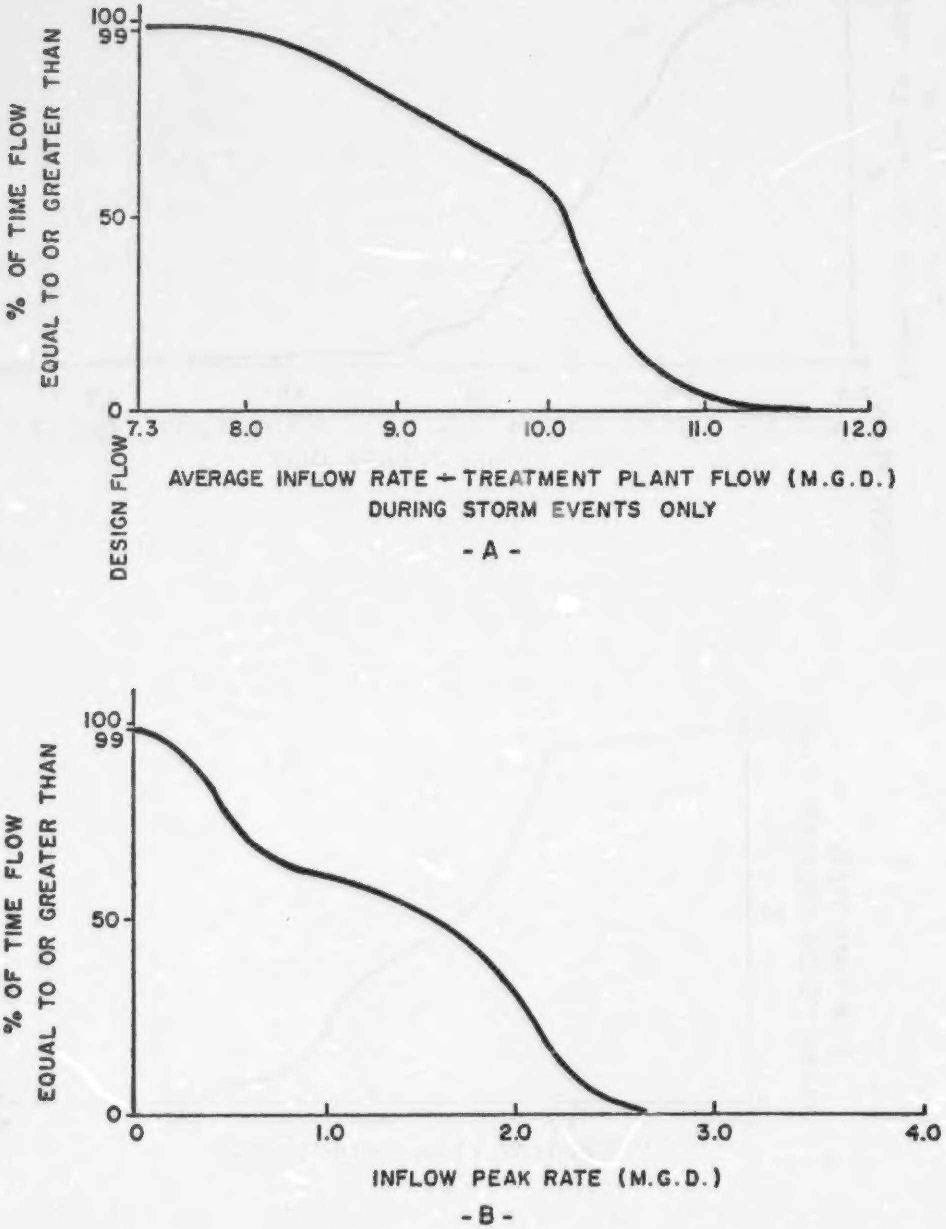
-A-

-B-

FIGURE NO. 3
IMPACT OF SUMMER INFLOW
VILLAGE OF WEBSTER

PHASE 2
INFLOW / INFILTRATION STUDY
OCTOBER 1977

FIGURE 4



IMPACT OF SUMMER INFLOW
TOWN OF WEBSTER

inflow could be handled with the hydraulic capacity of the dry weather facilities. It was then determined from the small time in which inflow occurred and the fact that the 'fixed' media filters would not be washed out as an activated sludge process might be during peak conditions that the overall effect on treatment was negligible. The phosphate removal facilities were found to be of adequate size, partially due to the 5 hr. 'lag' time for effluent from the Village to reach the Town's facilities, by which time Town inflows had already been treated and discharged.

The conclusion was that the O & M charges for treatment of inflow would be far less than identification and removal of the inflow (Table 1).

TABLE 1

Operation and Maintenance Costs of Treating Inflow

Process	Village Inflow including Snowmelt		West Webster Sewer District 3, Penfield Sewer District 3 Inflow	
	O&M/yr.	O&M Over Design Period*	O&M/yr.	O&M Design Period*
Phosphorus Removal	$1520.00	$15,283.00	$120.00	$1203.00
Disinfection	343.00	4,462.00	27.00	351.00
Totals	$1263.00	$19,740.00	$147.00	$1554.00

*Present worth figures based on 20-year period and interest rate of 7.0%

INTERLAKEN INFLOW ANALYSIS

The Village of Interlaken has a small sewer system and an overloaded treatment plant for which upgrading was planned. Future treatment facilities would include aerated lagoon treatment based on a design flow rate of .17 MGD and a 1.5 peaking factor. The lagoon system requires detention times of approximately twenty six (26) days to achieve the removal efficiencies required. This fact brought about the possibility of use of lagoon freeboard as an 'in-line' storm inflow storage technique. Also it was planned to pump from the lagoon to discharge at a controlled rate, easily allowing necessary variation of levels in the lagoon that would be more difficult to achieve with free discharge devices.

To analyze the impacts of storm inflow and weigh the costs of inflow removal against storage/treatment costs required implementation of 'STORM' for system statistical analysis for 10 years of rainfall record data. It is important to understand the importance of initial abstraction quantities and method of obtaining it. In Interlaken, as shown in Fig. 5, a rainfall comprising two consecutive peaks and their associated flows were analyzed to show the initial heavier rain produced less inflow than the subsequent rainfall, since most of it went to satisfy initial abstraction requirements and none of the following rain did. Simple comparison provided an initial abstraction value for the system as well as an imperviousness factor, later verified for other storms.

The results of the effort were profound. The total inflow volume over the design period was analyzed as to

VILLAGE OF INTERLAKEN N.Y.

FIG.5 EFFECT OF PRECIPITATION ON SEWAGE FLOW

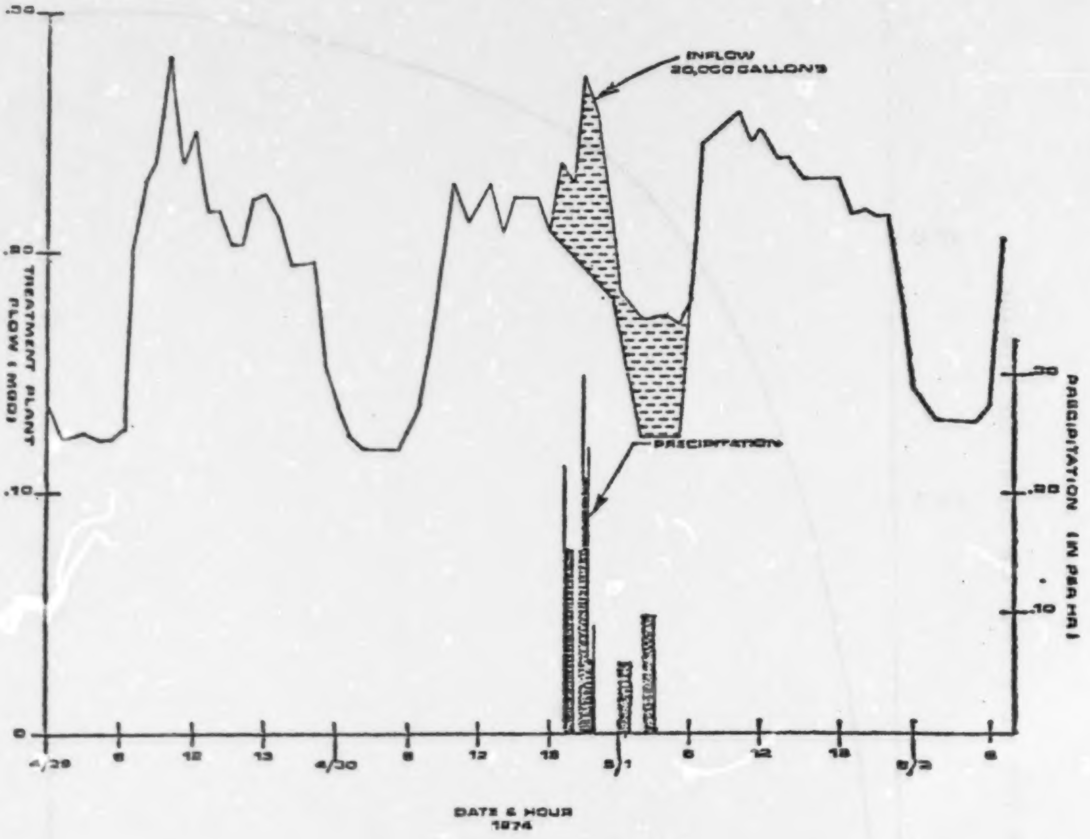
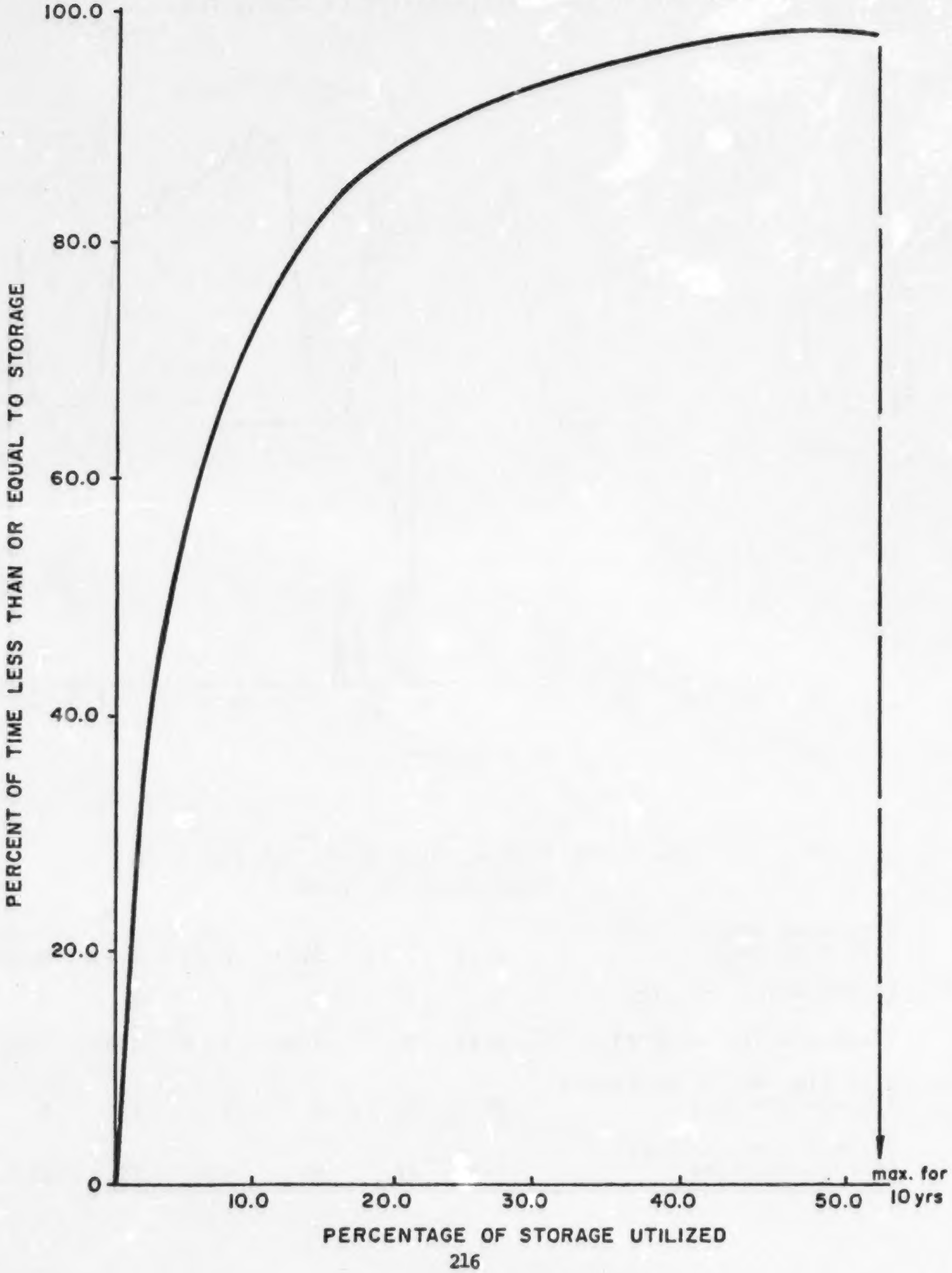


TABLE 2 STATISTICAL STORAGE UTILIZATION
DURING RAINFALL EVENTS

Treatment Plant Flow Rate (mgd)	0.17	0.17	0.17	0.25	0.25	0.25
Lagoon Freeboard (ft)	1	2	3	1	2	3
Freeboard Storage (mg)	0.45	0.90	1.35	0.45	0.90	1.35
% of Time 50% of Freeboard Storage Exceeded	20	10	8	6	4	0
Maximum % of Freeboard Storage Utilized	175	88	60	110	55	37

FIGURE 6 VILLAGE OF INTERLAKEN / EXISTING SEWER SYSTEM
NORMALIZED STORAGE UTILIZATION CURVE
TREATMENT RATE = .25 M.G.D. (PEAK RATE)
STORAGE CAPACITY = .90 M.G. (2' FREEBOARD)



its treatment costs, including O & M charges only. No extra design capacity or storage was to be needed, a conclusion derived from 'STORM' modeling, including a number of intense rainstorms, of the proposed facilities. The 'STORM' outputted storage utilization curve, Fig. 6, for the .25 M.G.D., .90 M.G. storage alternative (Table 2), depicts the use of freeboard storage, on a percentage basis, during all rainfall events. This handy, optional output allows the user to make maximum use of design storage at respective design flow rates for each alternative.

Cost comparisons of various storage/treatment alternatives then allow design based on the most cost-effective alternate to achieve the final goal. It is noteworthy that no increase in design flow capacity was needed for inflow removal with the storage available when modeling flow rates based on the peak flow factor, 1.5. The total annual costs associated with O & M for the proposed facilities were as follows for the .17 MGD design capacity:

POWER	$ 7,000
CHEMICALS	3,000
LABOR	12,000
TOTAL	$22,000

The average daily inflow was found to be .008 MGD and associated annual costs therefore .008/.17 x $22,000 = $1,035. The Phase 2 investigation of inflow alone was estimated at $52,700 or $4,975/hr. Amortized over 20 years at 7% interest.

The obvious conclusion was that inflow should be treated. The client was subsequently saved a needless costly investigation and rehabilitation program.

CANANDAIGUA, N.Y. INFLOW INVESTIGATION

Canandaigua is an older city of about 15,000 residents. New treatment facilities were being planned to accept a new county sewer system and relieve existing problems at the heavily overloaded sewage treatment plant. Preliminary design plans had been completed without the use of 'STORM' for inflow investigation and a recommendation for off-line inflow storage of 3 million gallon capacity had been made, based entirely on the conventional, accepted 'design year' rainstorm approach and total capture of corresponding inflow. 'STORM' modeling of inflow levels and treatment/storage/overflow alternatives (Table 3) in conjunction with a stream study of the receiving waters made it apparent that minor periodic plant overflows could be accepted if some degree of solids removal and disinfection were provided. A concurrent investigation of costs of various storage/treatment alternatives to achieve the same overflow removal rates made it apparent that storage was a less costly method of capturing inflow for subsequent treatment than simply increasing plant size without any storage (Table 4) above the design flow rate required for sanitary and cost-effective infiltration flows alone.

At this point the system was modeled based on a combination of storage/treatment alternatives and treatment

CANANDAIGUA, N.Y.

TABLE 3 TREATMENT / STORAGE / OVERFLOW ALTERNATIVES

Alternative

Treatment Rate MGD	Storage MG	No. of Overflow Events	Overflow MG	Time of Overflow Hr.	% Overflow Removal
6.5	0.5	74	215	1103	18
6.5	1.0	59	171	879	35
6.5	2.0	33	112	568	57
6.5	3.0	26	83	411	65
7.0	0.5	72	166	842	37
7.0	1.0	51	148	648	44
7.0	2.0	30	102	429	61
7.0	3.0	21	71	315	73

TABLE 4 TREATMENT / STORAGE / INFLOW REMOVAL COSTS

Alternative

Treatment Rate MGD	Storage MG	Treatment* $	Storage $	Total $	% Inflow Removal	$ x 10^6 $/Gal. Removed
6.5	0.5	500,000	400,000	900,000	18	0.050
6.5	1.0	500,000	600,000	1,100,000	35	0.031
6.5	2.0	500,000	1,000,000	1,500,000	57	0.026
6.5	3.0	500,000	1,300,000	1,800,000	65	0.027
7.0	0.5	1,000,000	400,000	1,400,000	37	0.039
7.0	1.0	1,000,000	600,000	1,600,000	44	0.036
7.0	2.0	1,000,000	1,000,000	2,000,000	61	0.033
7.0	3.0	1,000,000	1,300,000	2,300,000	73	0.032

* Cost increase from base treatment cost of $12,200,000 for 6 MGD Treatment Plant

CANANDAIGUA, N.Y.

TABLE 5 TREATMENT / STORAGE / OVERFLOW ALTERNATIVES WITH PEAK FLOW CAPACITY ALLOWANCE

Alternative					
Treatment Rate MGD	Storage MG	No. of Overflow Events	Overflow MG	Time of Overflow Hr.	% of Overflow Removed
6.5	1	59	171	879	35
6.5	2	33	112	568	57
10	1	25	58	188	78
10	2	16	37	125	86
16.5	1	8	18	46	93
16.5	2	4	9	26	97

TABLE 6 ALTERNATIVE COSTS / REMOVALS WITH PEAK FLOW CAPACITY ALLOWANCE

Alternative				
Treatment Rate MGD	Storage MG	Cost $	% Overflow Removed	$\$ \times 10^{6}$ % Overflow Removed
6.5	1	1,100,000	35	0.031
6.5	2	1,500,000	57	0.026
10	1	1,100,000	78	0.014
10	2	1,500,000	86	0.017
16.5	1	1,100,000	93	0.012
16.5	2	1,500,000	97	0.015

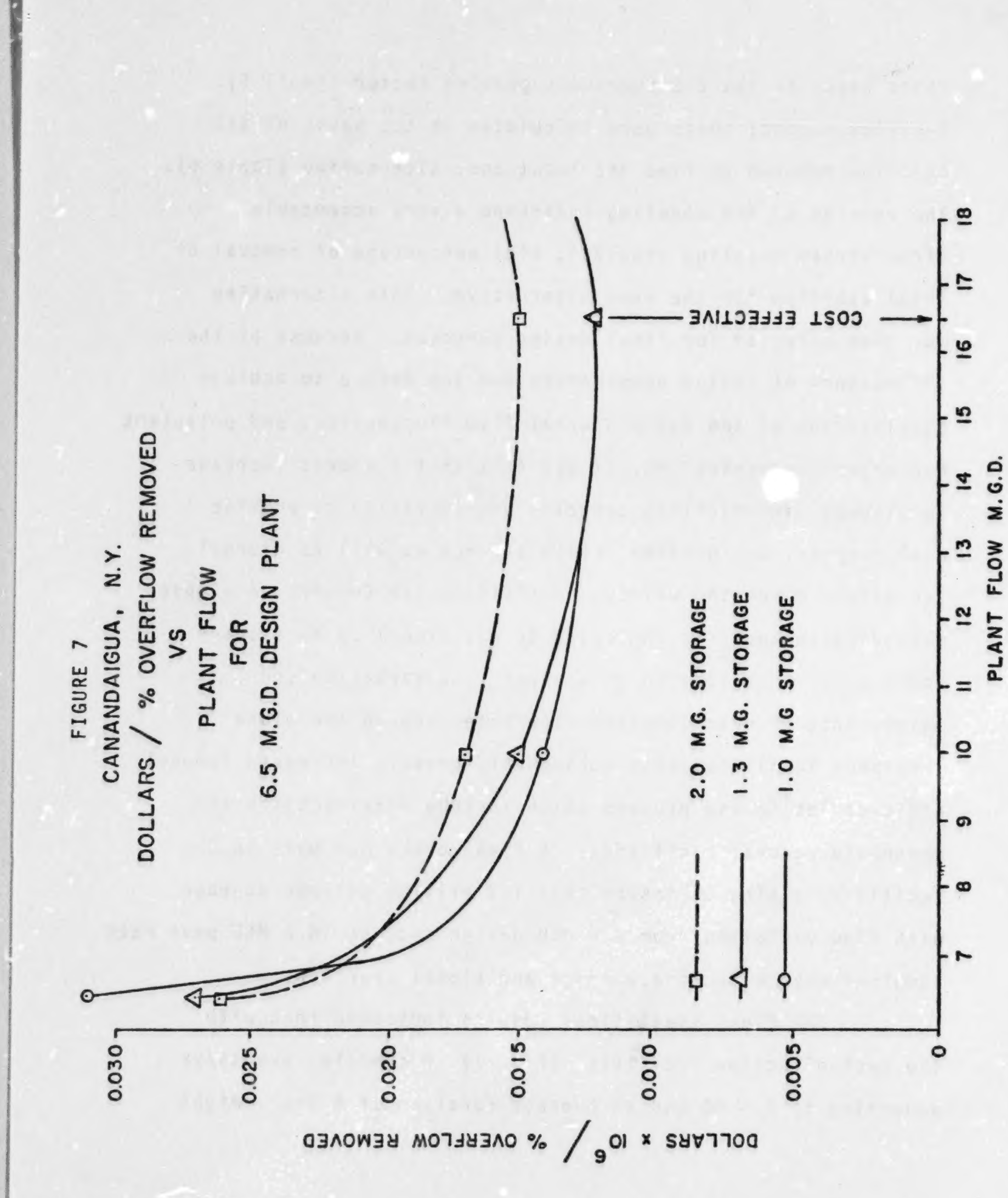


FIGURE 7

CANANDAIGUA, N.Y.

DOLLARS / % OVERFLOW REMOVED

VS

PLANT FLOW

FOR

6.5 M.G.D. DESIGN PLANT

rates based on the 2.5 hydraulic peaking factor (Table 5). Overflow removal costs were calculated on the basis of $/% overflow removed to find the least cost alternative (Table 6). The results of the modeling indicated a very acceptable (from stream modeling results), high percentage of removal of total overflow for the same alternative. This alternative was then selected for final design purposes. Because of the infrequency of inflow occurrences and the desire to achieve equalization of the daily diurnal flow fluctuations and pollutant concentration variations, it was felt that a modest increase in storage tank facility capacity was justified to provide dual purpose, wet weather inflow storage as well as diurnal variations dampening without sacrificing the former. A simple mass diagram analysis indicated an additional .3 MG storage would allow equalization of diurnal flow variation and maintenance of near constant flow rate through the plant treatment facilities with subsequent, greatly increased removal efficiencies in the process which include nitrification and phosphate removal facilities. A final check was made on the facilities sizing to insure that 1.3 million gallons storage with flow variation from 6.5 MGD design rate to 16.5 MGD peak rate remained cost effective without additional overflow.

The final statistical results indicated that with the cost-effective facilities (Fig. 7) .6 overflow events/yr., amounting to 1.4 MG and an average duration of 4 hrs., might be expected. Only 2.5% time, or less than 1 da./mo., is the flow rate expected to exceed the design flow rate. Again, 'STORM' allowed design of facilities at a great cost savings to the client over what other methods would have indicated was necessary.

SUMMARY & CONCLUSIONS

'STORM' may be used to model stormwater inflow into old, existing sanitary sewer systems as easily as for combined or separate stormwater sewers when the user properly selects and verifies his critical model calibration factors and basic assumptions.

The model, in conjunction with costs analyses and/or receiving stream studies, is a valuable tool at determining the cost-effective alternative in alleviating the problems associated with stormwater inflow. Great savings may be realized over what might be required by standard inflow investigation/system rehabilitation and the 'Design Storm' concept.

VERIFICATION AND CALIBRATION OF THE ILLINOIS URBAN AREA DRAINAGE SIMULATOR (ILLUDAS)

by F.I. Lorant and C. Doherty

M.M. DILLON LIMITED
CONSULTING ENGINEERS

SWMM USERS GROUP MEETING
OTTAWA, ONTARIO

May 1978

INTRODUCTION

This paper is a result of on-going research carried out by the staff of M.M. Dillon to calibrate and verify the Illinois Urban Drainage Area Simulator[1] (ILLUDAS).

ILLUDAS is a digital computer model which uses rainfall and physical basin parameters to predict runoff from paved and grassed areas.

This study utilizes recorded data collected by M.M. Dillon during a recent SWMM calibration and verification study[2] which was initiated by an Urban Drainage Sub-Committee set up under the Canada/Ontario and Canada/U.S. Agreement on Great Lakes Water Quality.

DESCRIPTION OF SELECTED URBAN CATCHMENT AREA

The selected urban catchment area is 383 acres in size, and is located approximately 4 miles to the northeast of downtown Toronto in the Borough of East York.

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[1] *The Illinois Urban Drainage Area Simulator, by Michael L. Terstriep and John B. Stall, Illinois State Water Survey, Urbana, Bulletin 58. 1974.*

[2] *Storm Water Management Model Verification Study, Borough of East York, by M.M. Dillon Limited, February 1978.*

The general surface topography is characterized by a gentle slope. The land use is predominantly residential (89.1%), the overwhelming majority of which is comprised of single family residential units, followed by institutional (5.7%), parks and open spaces (4.2%) and commercial (1.0%).

The total population within the catchment area was estimated to be in the region of 14,600.

The total area of impervious elements within the catchment area was estimated at 187 acres (49% impervious) of which approximately 52% is roads, sidewalks and driveways, 38% is roofs of houses connected to sewers, 4% is roofs of sheds not connected to sewers and 6% is miscellaneous.

RECORDED DATA

For the SWMM verification study (Borough of East York), M.M. Dillon collected and presented data on the following:

- Precipitation
- Runoff quantity and quality
- Air temperature
- Snow cover depth and water equivalent
- Municipal practices

The measurement of precipitation and the quantity of runoff was used to prepare storm hyetographs and hydrographs which were used to calibrate and verify ILLUDAS.

MODELLING

The main object of this study was to simulate the recorded rainfall runoff events with ILLUDAS.

The basic method used by ILLUDAS to compute runoff is as follows:

> Grassed and paved area supply rates are calculated by subtracting losses due to depressional storage and infiltration from the depth of rainfall. Next, a computation is made of the travel time required for each increment of runoff to reach the inlet followed by the derivation of a surface hydrograph. The surface hydrographs from each sub-basin are accumulated in a downstream order through the basin and routed through each section of pipe in order to account for the temporary storage within each pipe section. The result

is a computed outflow hydrograph for each section of pipe and ultimately a hydrograph at the outfall.

A more detailed explanation may be found in the ILLUDAS Users Manual, Bulletin No. 58.

DISCRETIZATION OF URBAN TEST AREA

A prerequisite to the preparation of input data is the discretization of the urban test area, i.e. the division of the area into overland flow areas and the sewer system into discrete elements for modelling purposes.

There is no ready rule for discretizing an area but the finer the discretization the more accurate can the rainfall runoff process be represented.

The discretization adopted for this study resulted in 33 sub-catchments ranging in size from 4.3 acres to 25.9 acres. The sewer system was represented by 33 conduits ranging in size from 18 to 66 inches. This resulted in 70% of the actual sewer system storage being modelled.

It should be noted that, due to the simplified approach used by ILLUDAS, several flow dividers which exist within the sewer system could not be modelled.

QUANTITY CALIBRATION

The ILLUDAS is designated as a "deterministic" model in that, if all input parameters are accurate, the physics of the processes are simulated sufficiently well to produce accurate results without calibration. In practice, however, many of the input parameters are not or cannot be accurately ascertained, thereby making it necessary to assume values which may not be representative of the study area. For this reason, a prerequisite to the verification of the ILLUDAS is the calibration process which entails the adjustment of input parameters until a good fit exists between the computed and measured hydrographs.

The input parameters that directly affect the quantity and peak flow rates are length and slope of overland flow, surface depressional storage, soil group, antecedent moisture conditions, amount of directly connected paved area and the amount of supplemental paved area.

For this study, the model was first calibrated for the directly connected paved areas and then the pervious areas. Three rainfall runoff events were chosen for the calibration procedure. The storm of July 7, 1976 was chosen to calibrate the directly connected paved areas and the storms of June 25, 1977 and June 29, 1977 were chosen to calibrate the pervious areas.

For the initial attempt at the calibration of the directly connected paved areas, it was decided that all the house roofs, all the roadways and all the sidewalks, approximately 153 acres, would be directly connected. This left all the driveways and all the sheds to be considered as supplemental paved areas approximately 37 acres. From hydrograph analyses and previous verification studies, the depressional storage was set equal to 0.02". These assumptions resulted in the simulated peak flow being within 2% of the recorded value and the simulated volume being within 0.5% of the recorded value. These results, which are shown in Figure 1, were considered acceptable, therefore no further calibration was attempted.

The next step entails the calibration of the pervious areas. Four input parameters affect the amount of runoff from the grassed areas. These are the supplemental paved areas since they run off instantaneously on to the grassed areas, the antecedent moisture condition, the depressional storage and the soil group. For this study, the supplemental paved area was considered to be the difference in the measured impervious area and the calibrated directly connected paved area. The antecedent moisture condition was determined from rainfall records. The remaining two values, depressional storage and the soil group were determined by adjusting these values until a good fit between the recorded and simulated hydrographs of June 25, 1977 and June 29, 1977 were achieved. These are shown in Figures 2 and 3 and resulted in a depressional storage of 0.1" and a soil group C being selected.

VERIFICATION

The verification included the simulation of 19 rainfall-runoff events. A summary of these events are presented in Tables 1 and 2. Results typical of these events are shown in Figures 4 to 11.

It should be noted that most of the recorded precipitation events during this collection period did not have sufficiently high intensities to overcome the modelled infiltration and depressional storage losses of the pervious areas. This meant that virtually all runoff originated from the impervious areas. Therefore it was not possible to verify ILLUDAS for runoff from the pervious areas.

The runoff volumes were simulated fairly accurately with the average and the standard deviation of the ratio of simulated to recorded values being 1.001 and 0.134 respectively, and the average and standard deviation of the ratio of simulated and recorded peak flow rates being 1.013 and 0.118 respectively.

About 84%, 53% and 26% of the simulated flow volumes were within 20%, 10% and 5% respectively of the record flow volumes. About 89%, 70% and 41% of the simulated peak flow rates were within 20%, 10% and 5% respectively of the recorded peak flow rates.

Since the aforementioned SWMM verification and calibration study was recently completed on this subcatchment with basically the same discretization scheme, the following is a comparison made between the results of these models.

Parameter	Number of Events	Mean of Sim/Rec		Std. Dev. of Sim/Rec	
		SWMM	ILLUDAS	SWMM	ILLUDAS
Peak Flow	31	1.016	1.026	0.116	0.111
Volume	15	0.970	1.019	.078	0.129

SUMMARY AND CONCLUSIONS

Recorded precipitation and urban runoff data has been used to calibrate and verify the Illinois Urban Drainage Area Simulator (ILLUDAS).

Initial results of the ILLUDAS output showed that both the hydrograph peaks and volumes generated by directly connected paved areas did not require any calibration. Depression storage and soil group parameters needed some adjustments for the calibration of pervious areas. Verification of 37 hydrographs produced by 19 rainfall-runoff events showed very encouraging results, indicated by the mean of 1.026 for the simulated and recorded peak flows and 1.019 for the simulated and recorded volumes.

Unfortunately, most of the monitored rainfall-runoff events had rainfall with relatively low intensities. As a result this study was only able to verify runoff from the pervious areas. However, the good results obtained do indicate that this relatively inexpensive computer model can be a fairly accurate tool for predicting the quantity of runoff from an urban catchment. The writers are confident that the same encouraging results will be obtained for higher intensity storms after calibrating the input parameters related to pervious areas.

TABLE 1

COMPARISON OF SIMULATED AND RECORDED PEAK RUNOFF

Storm Date			Simulated Peak Flow (cfs)	Recorded Peak Flow (cfs)	Ratio of Simulated to Recorded Peak Flows
Day	Mth.	Yr.			
27	03	76(I)	22.3	29.3	0.761
		(II)	24.3	25.5	0.953
31	03	76(I)	15.8	14.5	1.090
		(II)	17.0	17.0	1.000
		(III)	22.0	21.1	1.043
		(IV)	25.5	25.0	1.020
22	04	76	21.0	22.5	0.933
25	04	76(I)	23.3	20.0	1.165
		(II)	17.0	17.0	1.000
11	05	76	37.0	35.8	1.034
01	06	76	24.8	22.9	1.083
28	06	76	21.8	19.0	1.147
30	06	76(I)	40.1	37.0	1.084
		(II)	21.5	20.0	1.075
01	07	76	37.2	30.0	1.240
01	07	76	105.5	111.0	0.950
02	07	76(I)	12.0	10.0	1.200
		(II)	24.0	22.9	1.048
11	07	76	21.3	19.0	1.121
16	07	76	19.5	21.0	0.929
20	07	76(I)	26.7	24.0	1.113
		(II)	58.0	61.3	0.946
		(III)	52.7	55.2	0.955
29	07	76(I)	38.5	41.5	0.928
		(II)	31.5	29.5	1.068
31	07	76(I)	42.0	40.0	1.050
		(II)	19.8	20.0	0.990
		(III)	37.5	28.5	1.316
		(IV)	27.6	30.0	0.920
		(V)	23.2	23.2	1.000
		(VI)	25.8	26.6	0.970
13	08	76	100.0	121.0	0.826
01	09	76(I)	58.0	64.0	0.906
		(II)	22.5	22.5	1.000
04	09	76	55.0	68.9	0.798
10	09	76(I)	87.5	104.0	0.841
		(II)	64.0	66.7	0.960

Number of Peaks = 37

Mean of $\frac{\text{Simulated}}{\text{Recorded}}$ Peak Flows = 1.013

Standard Deviation of $\frac{\text{Simulated}}{\text{Recorded}}$ = 0.118

TABLE 2

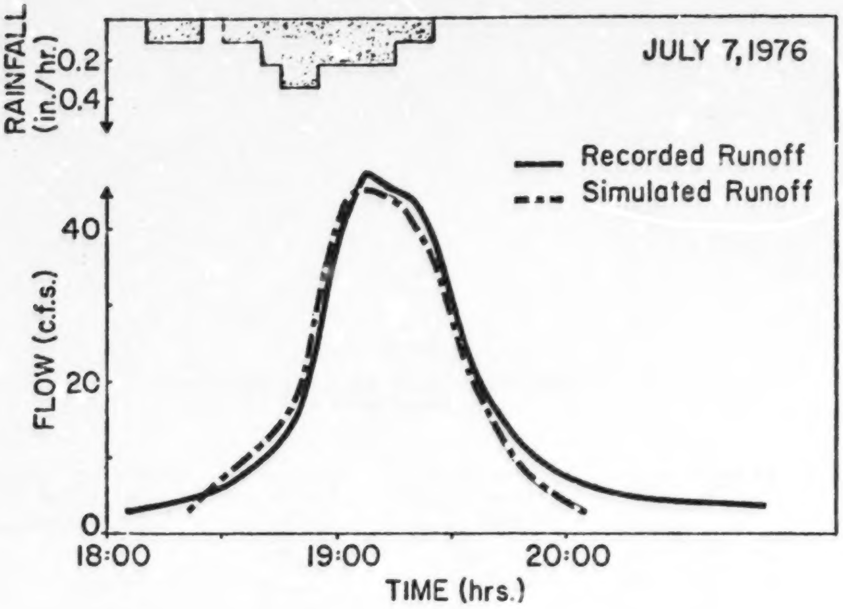
COMPARISON OF SIMULATED AND RECORDED FLOW VOLUMES

Storm Date			Simulated Flow Volume (cf)	Recorded Flow Volume (cf)	Ratio of Simulated to Recorded Flow Volumes
Day	Mth.	Yr.			
27	03	76	76,630	98,720	0.776
31	03	76	213,980	236,880	0.903
22	04	76	32,670	39,800	0.821
25	04	76	131,530	121,500	1.083
11	05	76	71,120	80,400	0.885
01	06	76	43,630	42,250	1.033
28	06	76	34,320	30,350	1.131
30	06	76	180,670	163,200	1.107
01	07	76	76,600	63,200	1.212
01	07	76	116,600	122,400	0.953
02	07	76	81,730	64,500	1.267
11	07	76	54,670	54,600	1.001
16	07	76	27,190	33,300	0.817
20	07	76	279,720	300,000	0.932
31	07	76	197,610	185,300	1.066
13	08	76	109,678	124,800	0.879
01	09	76	104,130	99,000	1.052
04	09	76	76,630	73,800	1.038
10	09	76	279,760	262,800	1.065

Number of Events = 19

Mean of $\frac{\text{Simulated}}{\text{Recorded}}$ Flow Volume = 1.001

Standard Deviation of $\frac{\text{Simulation}}{\text{Recorded}}$ Flow Volume = 0.134



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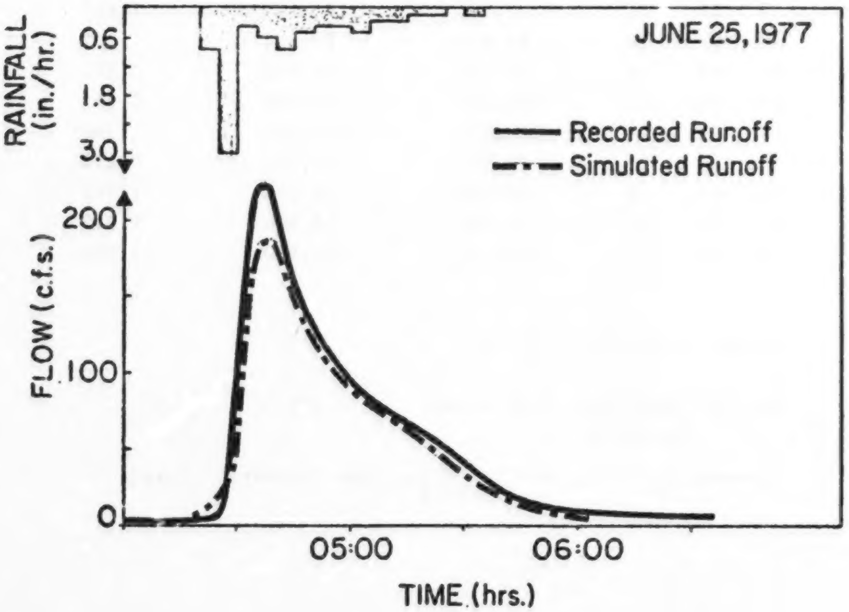
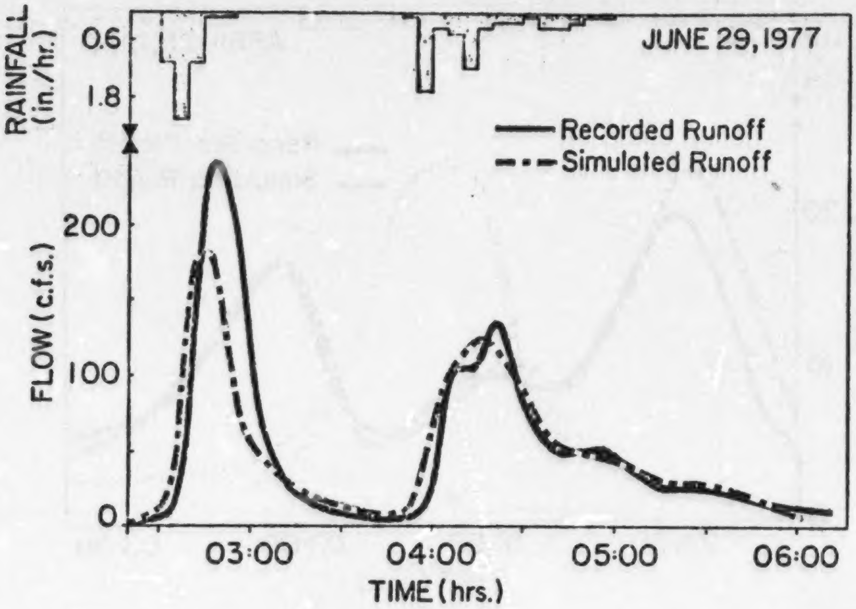


FIGURE 2



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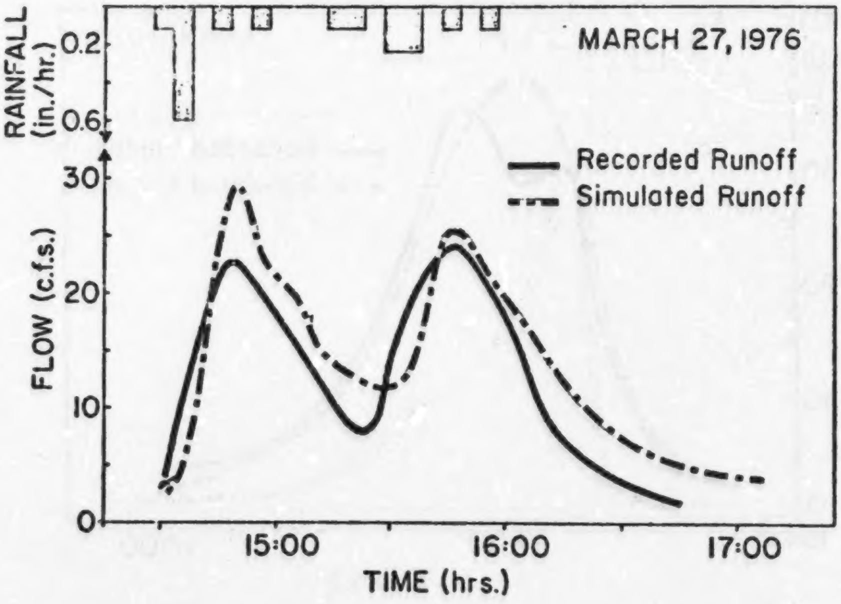

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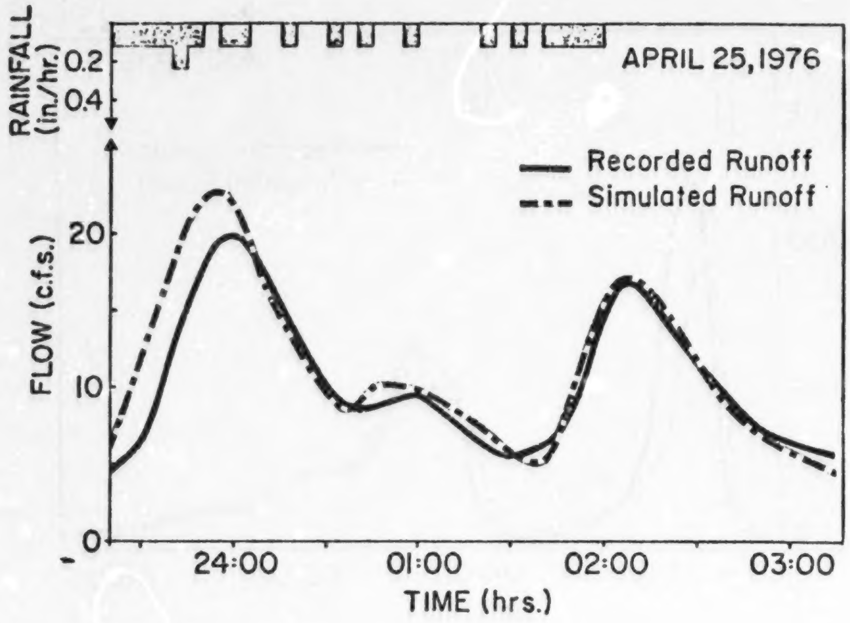

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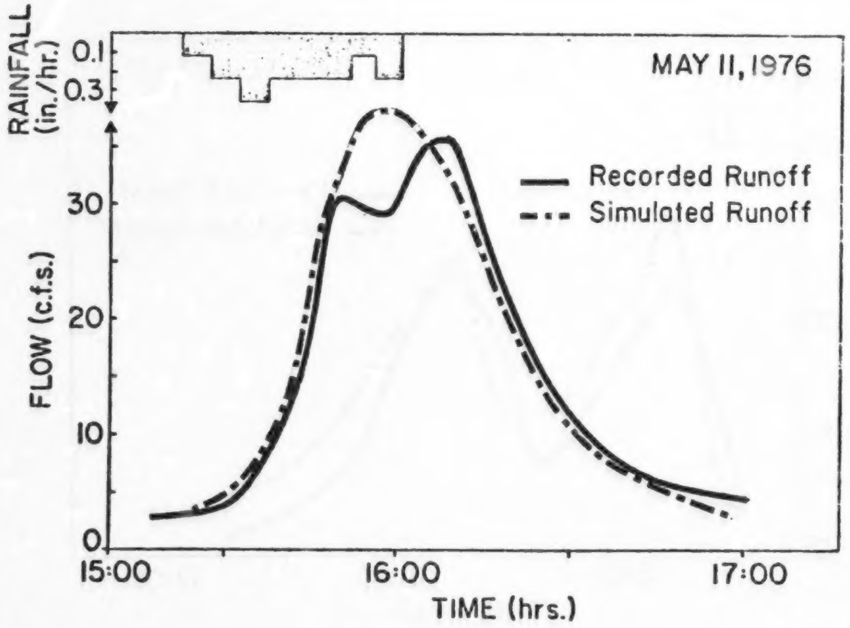

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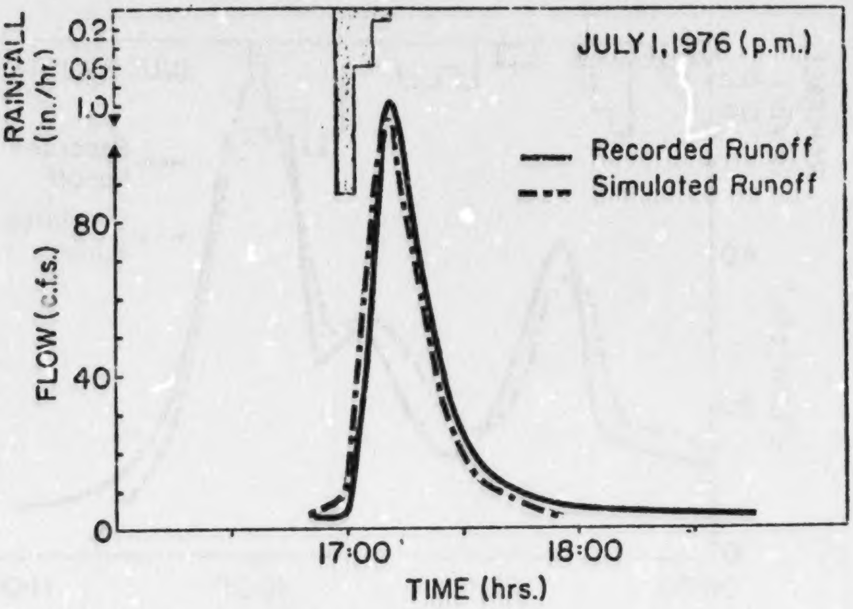

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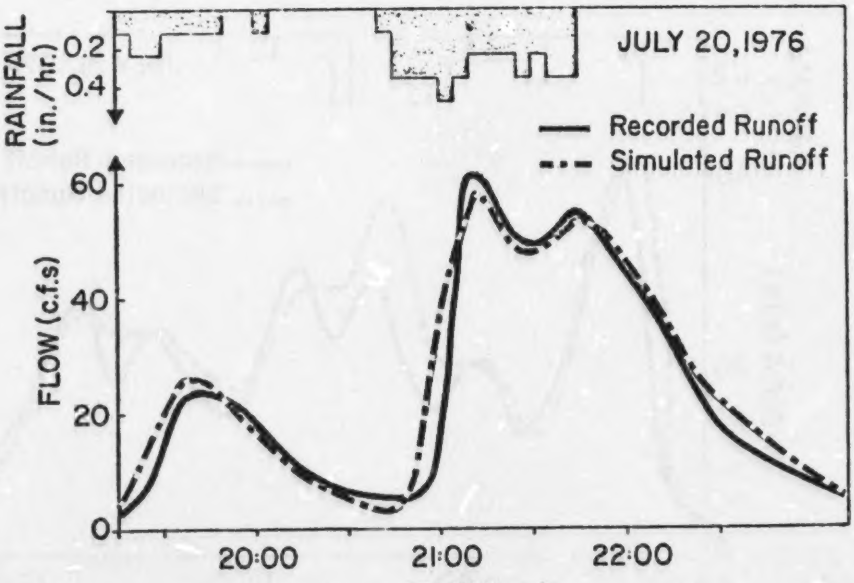

FIGURE 8

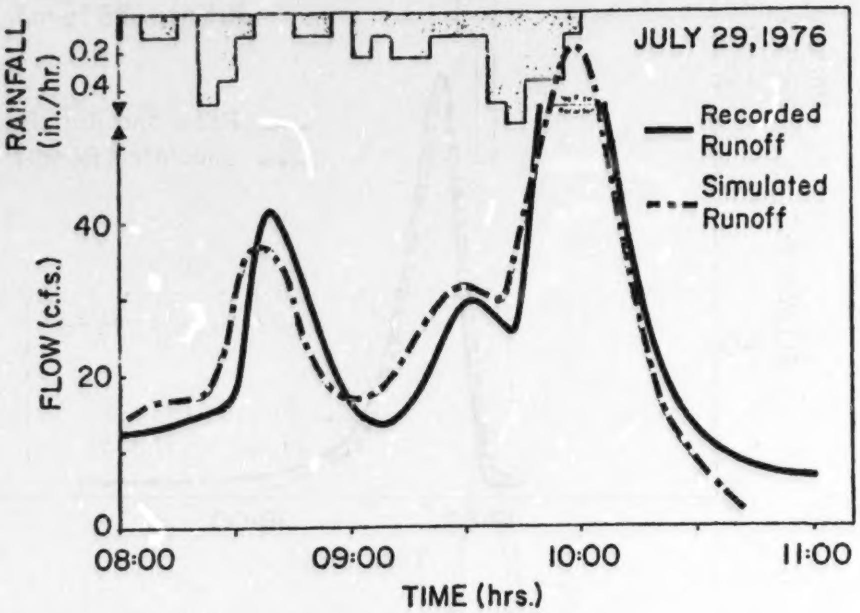

FIGURE 9

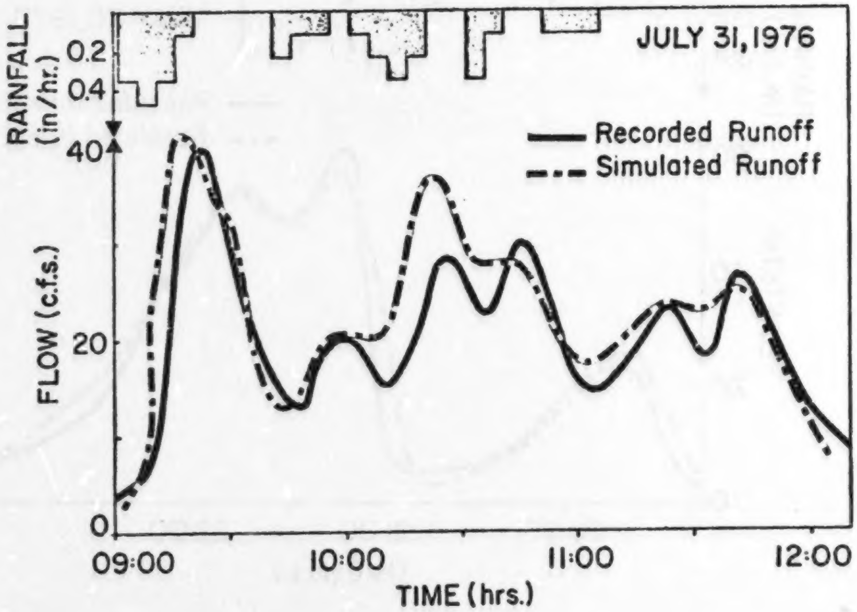

FIGURE 10

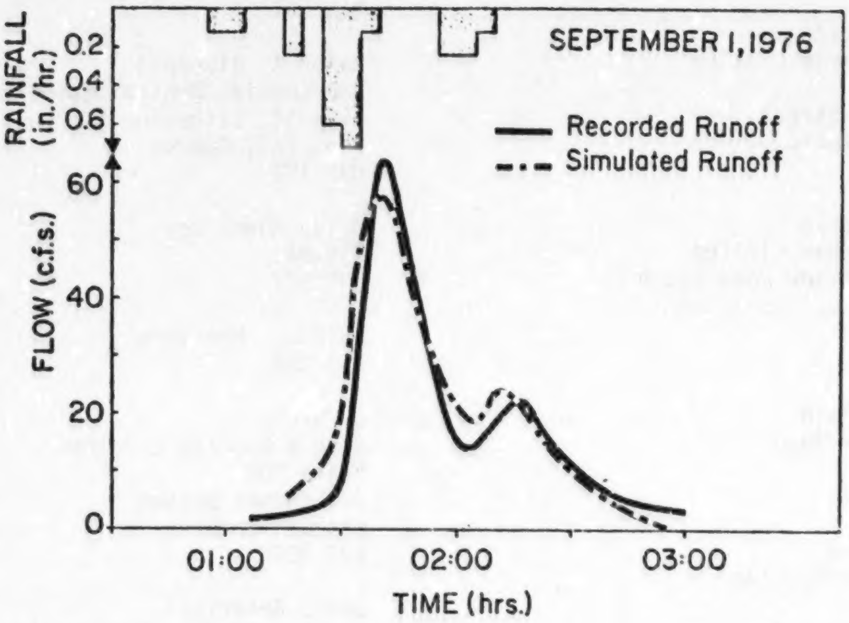
SEPTEMBER 1, 1976
RAINFALL (in./hr.)
0.2
0.4
0.6
FLOW (c.f.s.)
60
40
20
0
Recorded Runoff
Simulated Runoff
01:00
02:00
03:00
TIME (hrs.)

FIGURE 11

R. Adams,
Multiple Access Computer Group
#3 - 1479 Buffalo Place
Winnipeg, Manitoba
R3T 1L7

Quazi N. Alam
Totten Sims Hubicki Assoc. Ltd.
1A King Street East
Cobourg, Ontario
K9A 1K6

J. C. Anderson
Gore & Storrie Limited
Suite 700
331 Cooper Street
Ottawa, Ontario
K2P 0G5

Allan Arbuckle
R. E. Clipsham Limited
16 Mountainview Road South
Halton Hills (Georgetown)
Ontario
L7G 4K1

Albert T. Bain
52 Pann Ayr Road
Camp Hill
Pennsylvania

S. G. Barber
Ministry of Housing
6th floor
950 Yonge Street
Toronto, Ontario
M4W 2J4

Jean H. Bastien
City of St. Laurent
777 Laurentien Blvd.
St. Laurent, Québec
H4M 2M7

Raffi Bedrosyan
Borough of North York
5100 Yonge Street
Willowdale, Ontario
M2N 5N7

C. Bennett
City of Sarnia
City Hall
Sarnia, Ontario

Robert Berwick
Meta Systems
10 Holworthy Street
Cambridge, Ma.
02138

I. Bhatia
City of Ottawa
City Hall
111 Sussex Drive
Ottawa, Ontario
K1N 5A1

Ralph A. Bischoff
Beauchemin, Beaton, Lapointe
1134 St. Catherine St. W.
Montreal, Québec
H3B 1H4

Brian Bodnaruck
Box 86
GRP 327
R. R. #3
Selkirk, Manitoba
R1A 2A8

C. Brcic
Gore & Storrie Limited
Suite 700
331 Cooper Street
Ottawa, Ontario
K2P 0G5

David Brierley
De Leuw Cather, Canada Limited
133 Wynford Drive
Don Mills, Ontario
M3C 1K1

Eric Brown
Industrial Life-Technical Services Inc.
1202 - 101 Richmond Street West
Toronto, Ontario
M5H 1T1

Tas Candaras
Paul Theil Associates Ltd.
700 Balmoral Drive
Bramalea, Ontario

David J. Carleo
O'Brien & Gere Engineers Inc.
1304 Buckley Road
Syracuse, New York
13221

Ron Cebryck
City of Edmonton
City Hall
Edmonton, Alberta

Philip G. Clark
O'Brien & Gere Engineers, Inc.
1304 Buckley Road
Syracuse, New York

W. Clarke
James F. MacLaren Limited
320 Adelaide Street South
London, Ontario
N5Z 3L2

E. J. Cole
Township of Nepean
3825 Richmond Road
Ottawa, Ontario
K2H 5C2

Donald G. Cooke
Regional Municipality of Niagara
150 Berryman Avenue
St. Catherines, Ontario
L2R 7E9

G. Couchisne

W. J. Courtis
Walker, Newby & Associates Ltd.
10835 - 120 Street
Edmonton, Alberta

George V. Crawford
Gore & Storrie Limited
1670 Bayview Avenue
Toronto, Ontario
M4G 3C2

Jim Crowley
Ministry of the Environment
Environmental Approvals Branch
135 St. Clair Avenue West
9th floor
Toronto, Ontario
M4V 1P5

A. A. Dagostino
Corporation of the City of Sudbury
P.O. Box 1000
Sudbury, Ontario
P3E 4S5

T. Davis
Regional Municipality of Ottawa-Carleton
222 Queen Street
Ottawa, Ontario
K1P 5Z3

J. C. Dempsey
Department of Physical Environment
City of Ottawa
City Hall
111 Sussex Drive
Ottawa, Ontario
K1N 5A1

C. Doherty
M. M. Dillon Limited
50 Holly Street
Toronto, Ontario
M4S 2E9

R. Dunn
Ministry of the Environment
2378 Holly Lane
Ottawa, Ontario
K1V 7P1

C. W. Eicher
Gore & Storrie Limited
1670 Bayview Avenue
Toronto, Ontario
M4G 3C2

E. John Finnemore
Metcalfe & Eddy Inc.
1029 Corporation Way
Palo Alto
California
94303

Steven D. Freedman
Steams & Wheler
10 Albany Street
Cazenovia
New York
13035

J. R. Gamblin
Albery Pullerits, Dickson & Assoc.
29 Gervais Drive
Don Mills, Ontario
M3C 1Y8

Robert C. Ganley
O'Brien & Gere Engineers Inc.
1304 Buckley Road
Syracuse, New York
13221

R. Gietz
Regional Municipality of Ottawa-Carleton
222 Queen Street
Ottawa, Ontario
K1P 5Z3

Karl Gonnsen
City of Burlington
426 Brant Street
Box 5013
Burlington, Ontario

Martin H. Hawder
R. M. Kostuch Associates Ltd.
1417 C Cyrville Road
Ottawa, Ontario
K1B 3L7

David H. Hay
Environment Canada
Environmental Protection Service
Ottawa, Ontario
K1A 1C8

T. Hearn
City of Guelph
59 Garden Street
Guelph, Ontario
N1E 3P7

Karl Hemmerich
Dorsch Consult Limited
45 Richmond Street West
#1004
Toronto, Ontario
M5H 1Z2

Faris Hindie *
Ecole Polytechnique
University of Montreal
P.O. Box 6079
Succursale A
Montreal, Québec
H3C 3A7

G. Hicks

M. Hulley
St. Lawrence College
Cornwall, Ontario

Izal Hotzer
Polycom Systems
133 Wynford Drive
Don Mills, Ontario
M3C 1K1

L. Jandl
Ministry of Government Services
Downsview Computing Centre
1201 Wilson Avenue
East Building
Downsview, Ontario
M3M 1J8

Thomas K. Jewell
Department of Civil Engineering
University of Massachusetts
Amherst, Massachusette
01003

David Johnson
St. Lawrence College
Cornwall, Ontario

L. Kamp
Environment Canada
Environmental Protection Service
Ontario Region
Bogue Building
River Road
Ottawa, Ontario

W. L. Keay
Regional Municipality of Ottawa-Carleton
222 Queen Street
Ottawa, Ontario
K1P 5Z3

Ian Kennedy
J. L. Richards
864 Lady Ellen Place
Ottawa, Ontario
K1Z 5M2

Shiraz Khimji
Ministry of the Environment
985 Adelaide Street South
London, Ontario
N6E 1V3

---

* 1 day registration only

Bill Koeutzberger
Enuirex Inc.
5103 W. Beloit Road
Milwaukee, Wisconsin

Dennis F. Lai
Clinton Bogert Associates
2125 Center Avenue
Fort Lee, New Jersey
07024

Gerry Langman
Reid Crowther and Partners Ltd.
200 - 831 Portage Avenue
Winnipeg, Manitoba
R3G 0N6

Brian Larkin
Cumming Cockburn & Associates
5385 Yonge Street
Willowdale, Ontario
M2N 5R7

Dr. Guy LeClerc
Desjardins & Sauriol & Assoc.
1200 Boulevard St. Martin Ouest
Chomedey, Laval, Québec
H7S 2E4

W. Wayne Lee
Underwood, McLellan (1977) Ltd.
1479 Buffalo Place
Winnipeg, Manitoba
R3T 1L7

Pedro Liang
Corporation of the City of Kingston
Ontario Street
Kingston, Ontario

F. I. Lorant *
M. M. Dillon Limited
50 Holly Street
Toronto, Ontario
M4S 2E9

Jiri Marsalek *
Canada Centre for Inland Waters
867 Lakeshore Road
Burlington, Ontario
L7S 1A1

Barry McBride
James F. MacLaren
1240 Portage Avenue
Winnipeg, Manitoba R3G 0T6

Farrell McGovern
Oliver, Mangione, McCalla & Associates Ltd
1755 Woodward Drive
Ottawa, Ontario
K2C 0P9

M. B. McPherson
23 Watson Street
Marblehead, Massachusetts
01945

Dr. Miguel A. Medina Jr. *
Duke University
Durham, North Carolina

P. J. Meehan
Civil Engineering Department
University of Ottawa
Ottawa, Ontario

Constantin Mitici *
Ecole Polytechnique
University of Montreal
P.O. Box 6079
Succursale A
Montreal, Quebec
H3C 3A7

M. Monteith
Ministry of the Environment
2378 Holly Lane
Ottawa, Ontario
K1V 7P1

M. Nyugen Nam
2620 East Blvd. St Joseph
Montreal H1Y 2A4
Quebec, Canada

Van T. V. Nguyen *
Ecole Polytechnique
University of Montreal
P.O. Box 6079
Succursalle A
Montreal, Quebec
H3C 3A7

* 1 day registration only

Ronald W. Norton
Administrator
Engineering Services
Corporation of the Township of Nepean
3825 Richmond Road
Ottawa, Ontario
K2H 5C2

Z. Novak
Ontario Ministry of the Environment
Water Resources Branch
135 St. Clair Avenue West
Toronto, Ontario
M4V 1P5

Ron Ohman
McAuto
Box 5992
Berkeley, Montana
63134

T. J. Parent
W. L. Wardrop & Associates
77 Main Street
Winnipeg, Manitoba
R3C 3H1

Mario Parente
Gore & Storrie Limited
Suite 700, 331 Cooper Street
Ottawa, Ontario
K2P 0G5

H. Pascoe
City of Ottawa
City Hall
111 Sussex Drive
Ottawa, Ontario
K1N 5A1

A. Peltz
PRIME Computer Inc.
5801 Peachtree-Dunwoody Road
Atlanta, Georgia 30342

L. E. Pond
W. L. Wardrop & Associates
77 Main Street
Winnipeg, Manitoba
R3C 3H1

Rod Prosser
Lozier Engineers
50 ChestnutPlaza
Rochester, New York 14604

A. Puppa
Capital Region
Department of Public Works
19th floor
140 O'Connor Street
East Tower
Ottawa, Ontario
K2P 2H6

Pierre Purenne
Desjardins & Sauroil & Assoc.
1200 Boulevard St. Martin Ouest
Chomedey, Laval, Québec
H7S 2E4

Roger Quesnel
R. M. Kostuch Associates Ltd.
1417 C Cyrville Road
Ottawa, Ontario
K1B 3L7

Richard Race
Enuirex Inc.
5103 W. Beloit Road
Milwaukee, Wisconsin

V. A. Radzius
Transport Canada
Surface Structures Division (KMS)
Ottawa, Ontario
K1A 0N5

Jim Rawlings
Control Data Canada
130 Albert Street
Ottawa, Ontario
K1P 5G4

Louise Raymond
Bessette, Crevier, Parent, Tanguar & Assoc.
110 Ouest Boulevard Cremazie
Suite 1001
Montreal, Quebec
H2L 2G8

C. Riaz
University of Ottawa
Ottawa, Ontario

S. Sarikelle
Dept. of Civil Engineering
University of Akron
Akron, Ohio
44325

P. Sauvé
City of Ottawa
City Hall
111 Sussex Drive
Ottawa, Ontario
K1N 5A1

R. L. Scott
Lawson-Fisher Associates
101 JMS Building
South Bend, Indiana
46601

Dr. U. Shamir
42 Kirk Bradden Road East
Toronto, Ontario
M8V 2E7

Michael Shapiro
Dept. of City and Regional Planning
Harvard University
311 Gund Hall
Cambridge, Massachussets
02138

J. C. Simmonds
City of Ottawa
City Hall
111 Sussex Drive
Ottawa, Ontario
K1N 5A1

Ross Slaughter
Ministry of the Environment
135 St. Clair Avenue West
Toronto, Ontario
M4V 1P5

L. D. Smith
Suite 700 - 331 Cooper Street
Ottawa, Ontario
K2P 0G5

Guido S. Sondhe
Indiana State Board of Health
1330 W. Michigan
Indianapolis, Indiana
46206

L. South
Ministry of the Environment
133 Dalton Street
P.O. Box 820
Kingston, Ontario
K7L 4X6

A. Tafuri
U.S. EPA
Edison Waste Quality Res. Lab
Edison, New Jersey
08817

Paul Theil
Paul Theil Associates Ltd.
700 Balmoral Drive
Bramalea, Ontario

H. Torno
Environmental Protection Agency
Office of R & D (RD-682)
Washington, D.C.
20460

Clifford H. Tottle
City of Winnipeg
Water, Waste & Disposal Division
280 William Street
Winnipeg, Manitoba
R3B 0R1

J. Trottier

Ray Tugfar
Proctor & Redfern
75 Eglinton Avenue East
Toronto, Ontario
M4P 1H3

Paul S. Turje
Willie, Cunliffe, Tait & Co. Ltd.
827 Fort Street
Victoria, B.C.
V8W 1H6

Mr. A. Underhill
Twiga Consultants Limited
4800 Dufferin Street
Suite 200
Downsview, Ontario
M3H 5S9

David Utmeyer
Black & Veatch
1500 Meadow Lake Parkway
Kansas City, Montana

C. R. Walker
City of Ottawa
City Hall
111 Sussex Drive
Ottawa, Ontario
K1N 5A1

R. Warnock *
University of Ottawa
Ottawa, Ontario

D. Weatherbe
Ministry of the Environment
Water Resources Branch
135 St. Clair Avenue West
Toronto, Ontario
M4V 1P5

R. B. Wigle
James F. MacLaren Limited
435 McNicoll Avenue
Willowdale, Ontario
M2H 2R8

N. Wiggins
University of Ottawa
Ottawa, Ontario

R. Wilkinson
City of Ottawa
City Hall
111 Sussex Drive
Ottawa, Ontario
K1N 5A1

Paul Wisner
James F. MacLaren Limited
235 McNicoll Avenue
Willowdale, Ontario
M2H 2R8

Hsen Hseng Yeh
Envirosphere Co.
19 Rector Street
New York, New York
10006

G. Zukovs
Ministry of the Environment
Wastewater Treatment
Resources Road
Rexdale, Ontario

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